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THE FACTORS UNDERLYING THE RATIONAL DESIGN OF PAVEMENTS

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A. Analysis of the Pavement Design Problem.

When applied to a highway or airport pavement, the terms "satisfactory" or "adequate" cover a number of properties or characteristics. Thus, the satisfactory pavement must be relatively smooth, capable of carrying the necessary loads, economical, skid resistant and sufficiently durable to justify the investment. It is evident that there is no one simple criterion from which to judge the performance or adequacy of a pavement.

The attempts on the part of engineers to explore the reasons for various types of pavement failure and the efforts to control the numerous variables by means of specifications governing materials and construction, all have led to many pages of print and much discussion about the effects and influences of a large number of separate factors. So many aspects and phases have been discussed in engineering literature that there is need for clarification and understanding concerning the effect and relative importance of each of the numerous details.

An endeavor has been made herein to classify the important variables and to indicate the relationship between the properties of materials, effects of moisture, construction procedures, traffic and other modifying influences in order to show wherein each of these factors may affect some desirable property of the completed pavement. A chart, Figure 1, has been developed on the principle of subdividing or breaking down each item or property into the factors which have an influence upon that particular characteristic. It will be immediately evident that a number of very diverse elements are herein arranged in juxtaposition and the writer makes no claim that the arrangement is entirely adequate or consistently logical throughout. Nevertheless, it should serve in some degree to indicate the part played by each of the numerous details which when combined are responsible for the performance of a pavement so far as capacity to carry loads is concerned. It does not cover pavement disintegration or instability. This chart is arranged from left to right in order of increasing subdivision.

First, is the over-all problem involved in the question, "What type of pavement and base?" and "What is the minimum thickness which will be adequate and most economical for a given situation?" The first step in the analysis or breakdown is indicated

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in column two which contains a statement of the three primary problems that must be solved to produce a pavement that is structurally sound. In other words, the structural design of a pavement does not involve a single problem but requires a solution to three quite separate and distinct primary problems. Problem number one (column two) expresses the idea that an engineer must consider the relative permanency of the initial states of moisture and density in the soil at the finish of construction and base a design on the ultimate equilibrium conditions of the layers beneath the pavement--loose or compact--wet or dry.

Problem number two states the necessity for establishing conditions which will prevent plastic deformation of the basement soil. This involves the property of soils commonly referred to as bearing power or supporting value.

Problem number three involves the question of preventing the cracking and breaking up of the base and surface due to excessive flexing and bending over resilient foundations. This type of distress is more or less synonymous with "fatigue" failure.

These three primary problems are quite different in their essential nature and the engineer must employ different expedients to test the materials and to prevent or counteract unsatisfactory developments under each of these separate headings.

Column three of the chart is a list of the factors which are responsible for the variations in quality of the items listed under the primary elements.

Column four is a further breakdown or dissection of the factors and properties which make up the items in column two. Similarly, columns five and six list further subdivisions describing the numerous influences leading up to the principal items in preceding columns.

A chart of this type serves to clarify the question of materials testing and should make it quite evident that over-all performance tests can greatly simplify the problem of testing and evaluating all materials and factors. Briefly, the more nearly the test can be matched to the items in the left hand columns, the fewer are the tests which will be required.

However, limitations in laboratory test methods usually require a shift to the items of more limited scope. Discs and stars indicate the properties of pavement materials or of soils that are susceptible to direct testing or to calculated evaluation.

It, therefore, appears that at least ten properties or characteristics of basement soils, pavements and traffic must be evaluated or given consideration in order to carry out an intelligent and comprehensive design procedure.

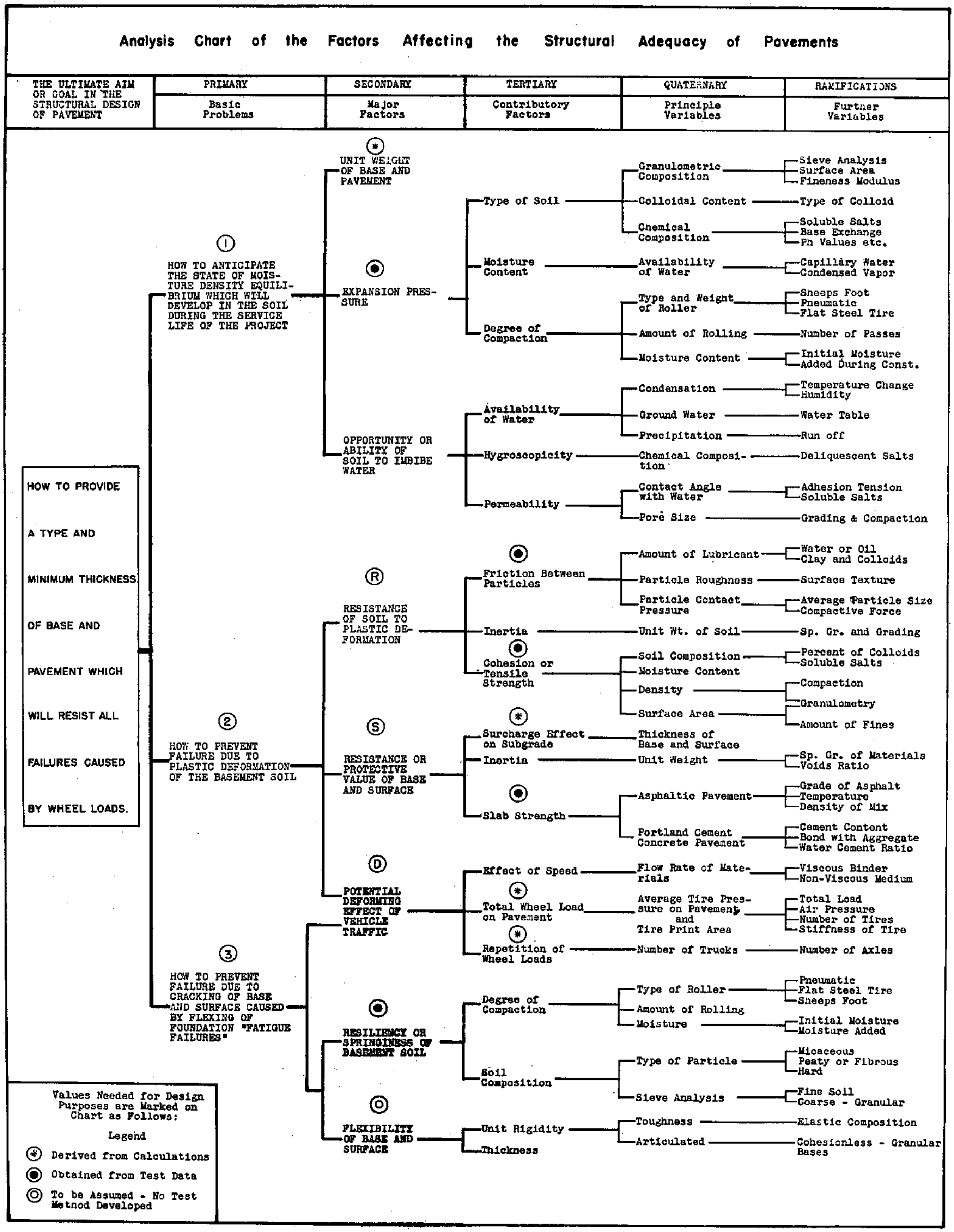
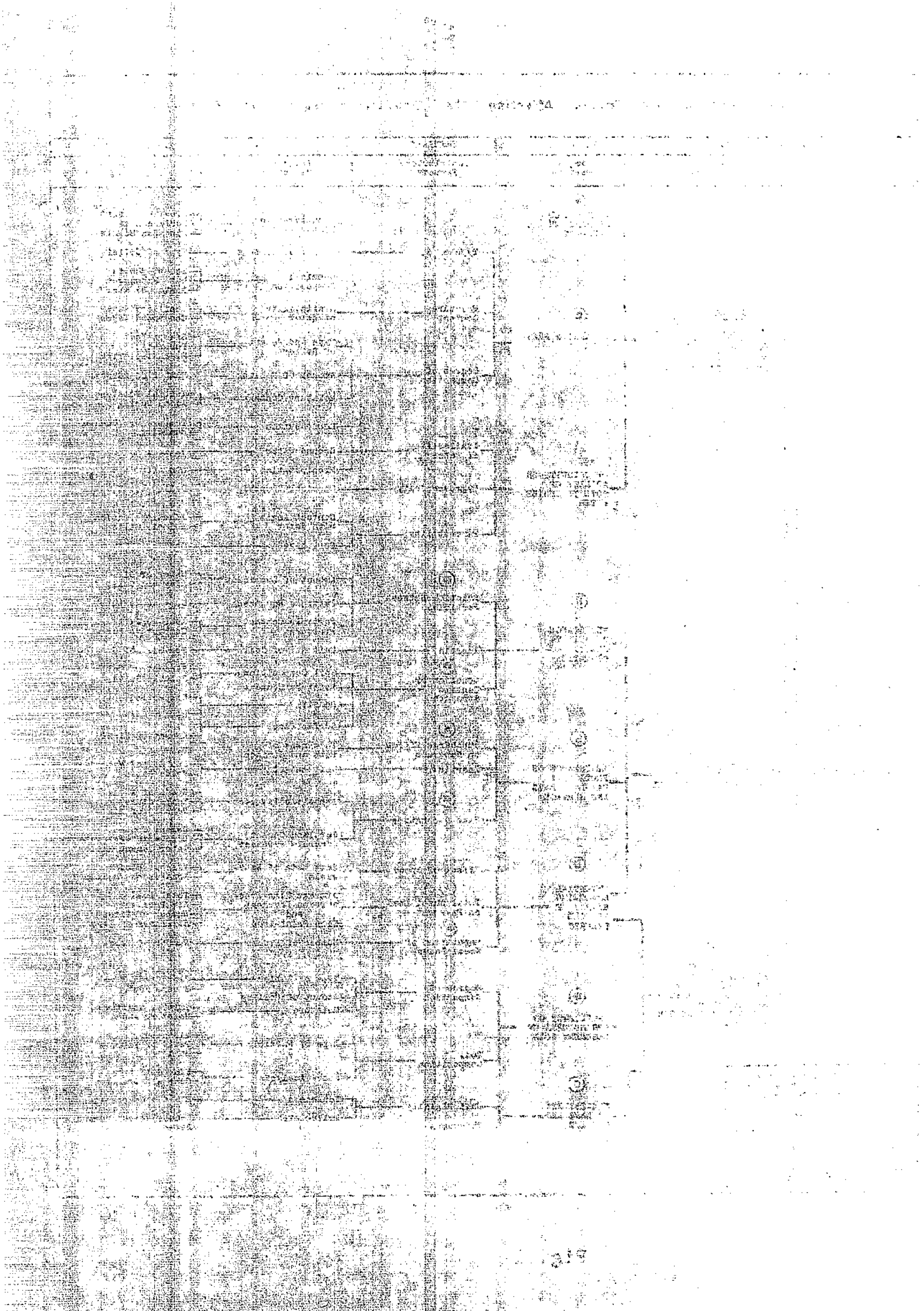

Analysis Chart of the Factors Affecting the Structural Adequacy of Pavements
THE ULTIMATE AIM OR GOAL IN THE STRUCTURAL DESIGN OF PAVEMENT
PRIMARY
Basic Problems
SECONDARY
Major Factors
TERTIARY
Contributory Factors
QUATERNARY
Principle Variables
RAMIFICATIONS
Further Variables
HOW TO PROVIDE A TYPE AND MINIMUM THICKNESS OF BASE AND PAVEMENT WHICH WILL RESIST ALL FAILURES CAUSED BY WHEEL LOADS.
1
HOW TO ANTICIPATE THE STATE OF MOISTURE DENSITY EQUILIBRIUM WHICH WILL DEVELOP IN THE SOIL DURING THE SERVICE LIFE OF THE PROJECT
UNIT WEIGHT OF BASE AND PAVEMENT
EXPANSION PRESSURE
Type of Soil
Granulometric Composition
Sieve Analysis
Surface Area
Fineness Modulus
Colloidal Content
Type of Colloid
Chemical Composition
Soluble Salts
Base Exchange
Ph Values etc.
Moisture Content
Availability of Water
Capillary Water
Condensed Vapor
Degree of Compaction
Type and Weight of Roller
Sheeps Foot
Pneumatic
Flat Steel Tire
Amount of Rolling
Number of Passes
Moisture Content
Initial Moisture
Added During Const.
OPPORTUNITY OR ABILITY OF SOIL TO IMBIBE WATER
Availability of Water
Condensation
Temperature Change
Humidity
Ground Water
Water Table
Precipitation
Run off
Hygroscopicity
Chemical Composition
Deliquescent Salts
Permeability
Contact Angle with Water
Adhesion Tension
Soluble Salts
Pore Size
Grading & Compaction
2
HOW TO PREVENT FAILURE DUE TO PLASTIC DEFORMATION OF THE BASEMENT SOIL
R
RESISTANCE OF SOIL TO PLASTIC DEFORMATION
Friction Between Particles
Amount of Lubricant
Water or Oil
Clay and Colloids
Particle Roughness
Surface Texture
Particle Contact Pressure
Average Particle Size
Compactive Force
Inertia
Unit Wt. of Soil
Sp. Gr. and Grading
Cohesion or Tensile Strength
Soil Composition
Percent of Colloids
Soluble Salts
Moisture Content
Compaction
Density
Granulometry
Surface Area
Amount of Fines
S
RESISTANCE OR PROTECTIVE VALUE OF BASE AND SURFACE
Surcharge Effect on Subgrade
Thickness of Base and Surface
Inertia
Unit Weight
Sp. Gr. of Materials
Voids Ratio
Slab Strength
Asphaltic Pavement
Grade of Asphalt
Temperature
Density of Mix
Portland Cement Concrete Pavement
Cement Content
Bond with Aggregate
Water Cement Ratio
D
POTENTIAL DEFORMING EFFECT OF VEHICLE TRAFFIC
Effect of Speed
Flow Rate of Materials
Viscous Binder
Non-Viscous Medium
Total Wheel Load on Pavement
Average Tire Pressure on Pavement and Tire Print Area
Total Load
Air Pressure
Number of Tires
Stiffness of Tire
Repetition of Wheel Loads
Number of Trucks
Number of Axles
3
HOW TO PREVENT FAILURE DUE TO CRACKING OF BASE AND SURFACE CAUSED BY FLEXING OF FOUNDATION "FATIGUE FAILURES"
RESILIENCY OR SPRINGINESS OF BASEMENT SOIL
Degree of Compaction
Type of Roller
Pneumatic
Flat Steel Tire
Sheeps Foot
Amount of Rolling
Moisture
Initial Moisture
Moisture Added
Soil Composition
Type of Particle
Micaceous
Peaty or Fibrous
Hard
Sieve Analysis
Fine Soil
Coarse - Granular
FLEXIBILITY OF BASE AND SURFACE
Unit Rigidity
Toughness
Elastic Composition
Thickness
Articulated
Cohesionless - Granular Bases
Values Needed for Design Purposes are Marked on Chart as Follows:
Legend
Derived from Calculations
Obtained from Test Data
To be Assumed - No Test Method Developed

Fig. 1



In parts three and four of this paper, procedures will be suggested for dealing with problems number one and two. A rational solution for problem three is yet to be worked out. Fortunately, however, failures under the third primary group are somewhat in the minority and resilience of the basement soil is probably the least serious cause for distress compared to the importance of the other two considerations. It cannot be dismissed however.

It is believed that the chart is otherwise self-explanatory and while individual engineers may not agree in all respects with the arrangement and allocation of factors, nevertheless, it is hoped that it will serve to indicate the reasons for approaching the problem in the fashion developed in the following text and the reasons why certain tests are proposed and perhaps, most important, should make it clear that the test methods, formulas and design charts submitted herewith are applicable only to problems number one and two and are not considered as final or as capable of completely solving all problems of pavement design.

A study of the chart, Figure 1, indicates that there are a number of items or subjects which could logically be taken up for further discussion; for example, the test equipment required or the effects of the numerous variables. However, inasmuch as the principal matter under consideration is the nature and behavior of soil and the ability to support loads, it seems most appropriate to next consider the nature and characteristics of the soil materials in order to discover what properties must be determined by test.

B. <u>Behavior patterns developed in masses of granular materials under load.</u>

For a very long time engineers have been faced with the problem of utilizing soils and granular materials in engineering works. It is only stating the obvious to say that soils and various combinations of rock, sand and gravel are the oldest of engineering materials. It does, however, seem a little strange that such a commonplace substance could be the cause for so much discussion and even argument. For example, after years of road building and volumes of theoretical studies there still seems to be much difference of opinion concerning the fundamental principles and the methods to be used in computing the ability of soils to support highway and airport pavements under heavy wheel loads. This, of course, does not mean that all past construction has been inadequate. Many successful pavement and base combinations have been built as it is always possible to add increased thickness and strength to cover any uncertainties in the design concept. Thus, by the time-honored process of making it "hell-for stout", most installations can be made to stand up. However, extravagant over-design is not engineering.

The materials of the earth's crust which, for engineering purposes, are often described under the single heading of "soil", present many aspects. Having been utilized by mankind for an infinite variety of purposes, the knowledge of soils and related materials has been developed individually and variously by those interested in geology, mining, agriculture and in the ceramic arts, as well as by those concerned only with engineering works. Engineers have borrowed from the studies of the agricultural experts much of the terminology and certain of the test methods. It is also evident that the technology of clays and molding sands can be studied with profit.

It is probable that military engineers were concerned with the stability of earth works as far back as were the civil engineers and one of the oldest formulas expressing the limit of equilibrium for a soil mass was derived by a military engineer, Charles Augustin Coulomb, who lived between the dates of 1736 and 1806. Coulomb was concerned with the stability of embankments supported by brick retaining walls and his well known equation states that the resistance of a soil mass to sliding on a given plane is equal to

$$S = C + p_n \tan \phi$$

C = Cohesion

p_n = Pressure normal to the sliding plane

ϕ = Angle of friction

Since Coulomb's time, a great deal of the theoretical work on soils mechanics has leaned heavily upon the mathematical approach. The problem seems to have a special appeal to those who are able to handle complicated mathematical relations with ease and facility. This mathematical approach to the problem is commonly referred to as a "fundamental approach" but by the nature of the process certain assumptions must be made; therefore, it is customary in a mathematical analysis of soil behavior under load to regard soil as an idealized uniform substance which has been described as "elastically isotropic and monotonously homogenous".

One of the early mathematicians appeared to recognize that there might be some discrepancy between this ideal hypothetical substance and actual soil. J. J. Sylvester, (1), a contemporary of Rankine, served notice in the opening paragraphs of his presentation that he wished it to be understood that he was dealing with "mathematical soil" and not real soil.

The average highway engineer who must build a road cannot ordinarily accomplish much if he must find "ideal soil" for his embankments and foundations. Unfortunately, most real soils are not monotonously uniform and the degree of compaction and the moisture content are variable. Vehicle load applications are fleeting and transient and there is a discrepancy between the conditions assumed in most theoretical analyses and the actual conditions which commonly exist in granular bases or in the underlying soils of highway and airport projects.

In contrast to the approach which stresses the importance of "fundamentals", there is the alternate procedure which involves the experimental method which, to be successful, requires close observation of the actual behavior of materials under the conditions of service then the development of a theory that will embrace and explain all of the known facts and finally laboratory tests must be devised which will subject the materials to stress conditions similar to those in the prototype and thus make it possible to assign numerical values to the significant properties. This process represents a combination of empirical data analyzed in the light of known laws governing the behavior of matter and involves both theoretical concepts and direct observation. However, there are those who feel that in order to be "respectable" any theory or analysis must be sanctified by proper mathematical treatment.

One of the most difficult hurdles to be surmounted in the presentation of a viewpoint is the choice of language and terminology for the accurate conveyance of ideas or concepts. It is a common experience to find that heated arguments and apparently irreconcilable differences of opinion can be composed once there is an agreement on terminology and the meaning of words. Therefore, the following explanations are included as an essential step in setting forth the viewpoint. These are not presented as comprehensive definitions but are rather intended to focus attention on certain special aspects or implications of the terms that are pertinent to this discussion.

Soil. The word should be classed as a collective noun. It describes a mass or agglomeration composed of separate particles of mineral aggregates ranging in size from extremely fine to very coarse. For engineering purposes soil is not limited as in the agricultural sense but it should be emphasized that any soil regardless of the degree of fineness is not, strictly speaking, a single material; it is a mass or collection of particles of materials.

Soil Mechanics. The mechanics of granular materials involving interfacial relationships between solid particles under varying conditions of pressure and consolidation with or without the presence of lubricating liquids and colloidal complexes.

Basement Soil. This term is intended to cover the variable depth of material involving both cut and fill sections, below the subbase or base. The depth of the basement soil will be variable but may be considered as limited to the depths of the fills.

Stability. A term that has many meanings such as stability of chemical solutions, stability of retaining walls - in this paper it means ability of a soil mass or bituminous pavement to resist plastic deformation under repeated stress conditions developed by vehicle traffic or intermittent loads.

Bearing Power (Or Supporting Power). Has specific meaning for soils only when load, moisture and compaction, unit pressure, load area, and time are specified. As commonly used, it has no specific meaning. One might as well ask "how much load will a beam support"?

Failure. As ordinarily applied to test results on soils, the term is indefinite and has significant meaning only when all conditions are defined. It may mean crushing, rupture or simply plastic deformation. The term "failure" is misleading when used to describe plastic deformation.

Friction. May apply to solids or liquids. The resistance due to solid particle friction will vary in magnitude depending on surface roughness, amount of pressure and degree of lubrication. The internal friction of liquids is usually designated as viscosity, and the resistance varies with speed and area.

Lubrication. The process whereby friction is diminished through the effect of a liquid or other substance which is in place between the primary solid surfaces. One of the most important single factors affecting the stability of soil masses and a term which appears but rarely in soil mechanics literature.

Cohesion or Tensile Strength in soils is a property of moisture films generally increasing with large surface areas furnished by fine particles such as clay. In bituminous mixtures cohesion is supplied by the asphalt. Variability in cohesion or tensile strength in soils parallels the behavior of liquid friction in that the speed of action, surface area and temperature cause similar variations.

A great many aspects of the ability of soils to support loads have been described and illustrated by numerous investigators and it appears that certain generalizations can be made based on the evidence of a few of these illustrations that are reproduced here. In Public Roads, June, 1930, C. A. Hogentogler gave the following illustration "Sketch showing the supporting power of cohesionless sand under load areas of one square foot and ten square feet respectively." Figure 2. (It is understood that these sketches represent factual data derived from experiments with brick pavements in Florida.)

These diagrams warrant the conclusion that there is a marked difference in behavior of soils as load areas are varied (Goldbeck (2)) and indicate that the difference in behavior depends upon whether the resistance to deformation is due primarily to internal friction or to cohesion of the liquid films. It is, furthermore, evident that when friction between particles is a tangible element the supporting power of a granular mass can be enhanced by a surcharge over the upper surface beyond the boundary of the loaded area. On the other hand, where the internal friction is low (as in wet clay) and such resistance as exists is chiefly due to cohesion (liquid friction) then a surcharge adds to the total supporting capacity an amount equivalent to little more than the weight of the surcharge. Therefore, that portion of the total resistance and only that portion which arises from internal friction can be amplified by means of restraining forces in the form of either a weight surcharge or in the form of cohesive resistance beyond the confines of the area under load (or both). Fig. 3. Hence, no rational conclusion can be reached from test values which combine the effects of both friction and cohesion in unknown proportions. For example the C.B.R., unconfined compression, Marshall test, Hubbard-Field, etc.

Sand. Figure 4 is a photograph of a sand model (3) which indicates the existence of a fixed angle associated with a sharply defined plane limiting the boundary of active pressure. This angle is shown to be constant for the given sand material regardless of the type of movement of the restraining wall.

Extending the evidence from Figures 2, 3 and 4, to the form shown in Figure 5, the lines A,B,C represent the outlines of an undisturbed "core structure" in a mass of cohesionless sand supported only on a level base while the lines AD and AE represent a typical angle of repose of 34 degrees. Therefore, it appears that the shaded area A,C,E, may be regarded as a surcharge or counter weight required to maintain the basic triangular "structure" in a state of equilibrium. If a small cone or triangular prism (AFG), Figure 6, having a base area of one square foot and weighing approximately 24 pounds$\pm$ is removed and replaced by a weight of 770 pounds (corresponding to the load in Figure 2a) it then becomes necessary to supply an additional surcharge (Figure 7) which amounts to flattening the slope of Figure 6 until it reaches the horizontal plane, and thus establishing or duplicating the conditions of equilibrium shown in Fig. $3a_1$. Any additional increase in load must be likewise counter balanced by a proportional surcharge, Figure 8. Figure $3a_2$. The reasons for this relationship between load and lateral support are not hard to trace if we consider other data illustrating the behavior of materials.

Further evidence has been secured by observing the movement of sands and clay soils in boxes equipped with a heavy glass plate on one side. Photographs have been taken by the method of continuous exposure of the film which clearly indicate the direction of particle movement of either sands or clay particles.

Figure 9 shows the flow of sands both to right and to left of the load cross section. This simultaneous flow in two directions is somewhat difficult to achieve in the model. Conditions in a more typical case are shown by Figure 10 in which, for the period of recording, the flow followed a distinctly curved pattern in one direction.

Figure 11 illustrates the characteristic movement under loads placed upon an inclined slope.

Figures 9, 10 and 11 are not considered to reveal anything not previously known and more or less confirm evidence produced by other investigators. However, Mr. C. F. Kettering (4), pointed out in one of his papers that it is often helpful in studying a problem for the observer "to stand on his head for awhile". Figure 12 illustrates the appearance presented by these sand deformation tests if the camera is stationary with reference to the loading block, thus giving the appearance that the mass of sand is moving upward. It must be emphasized that Figures 9 and 12 represent the same phenomena viewed from a different point of reference. The contrasting appearance reminds one of the old saying that "what you see depends upon where you stand".

Professor Housel (5), presented photographs of a clay mass being deformed under load, Figure 13 and as pointed out by Mr. Vokac (6) the pattern developed by either sands or clay is very similar. In addition to the photographs shown in Figures 9 to 13, other observations have been made directly upon models and the different effects observed are illustrated in four sketches, Figures 14 to 17.

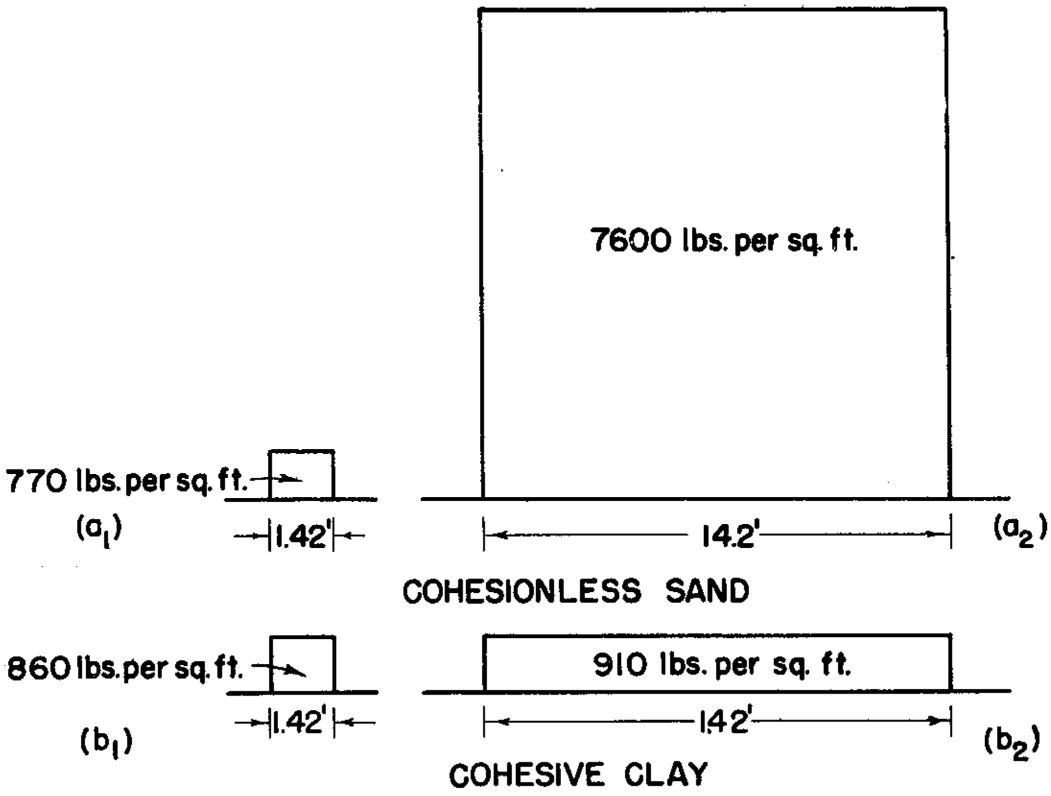


Fig. 2

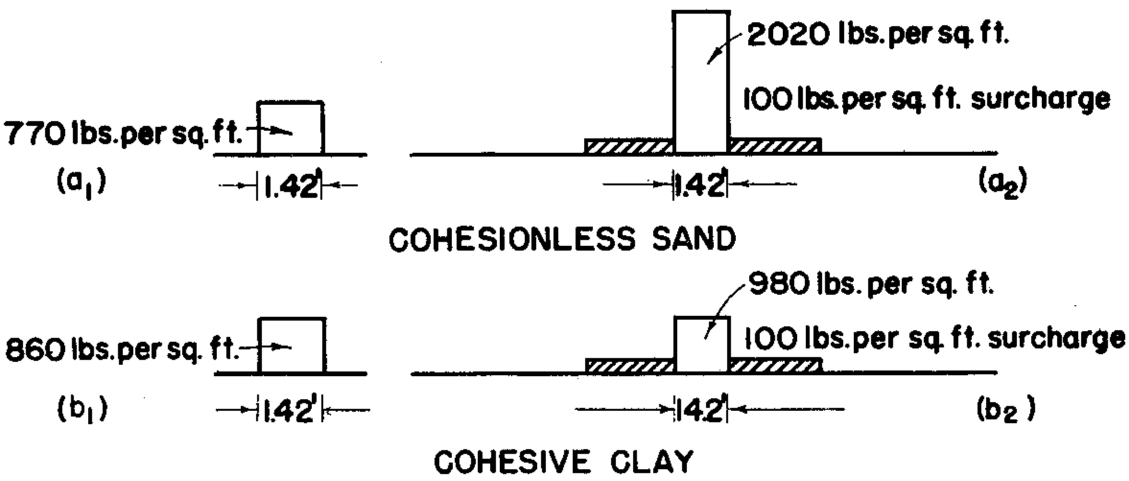
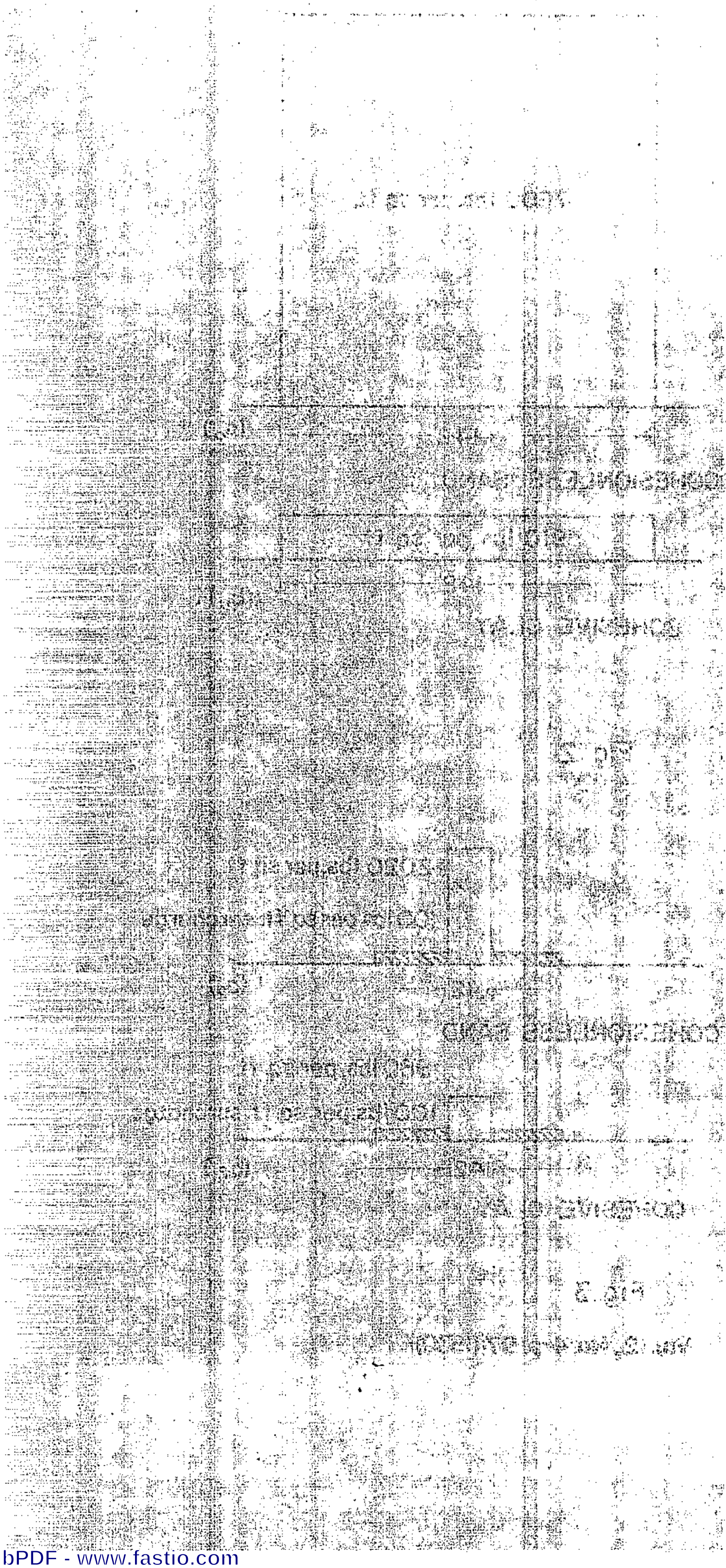
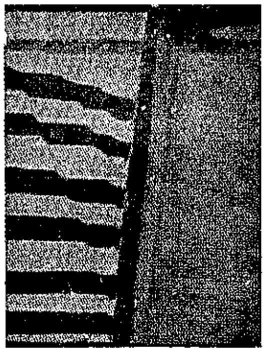


Fig. 3

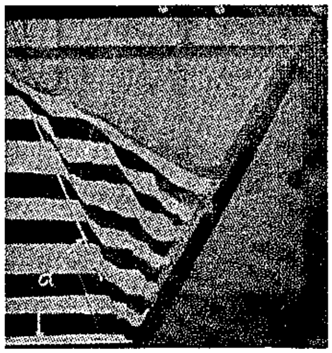
From Public Roads Vol. 12, No. 4 p. 97 (1930)



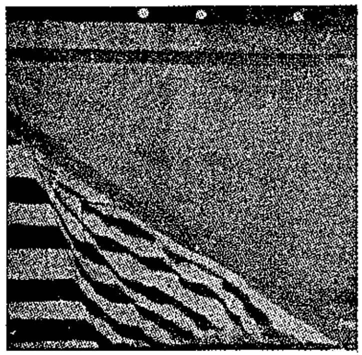
THE INTERNAL MOVEMENT OF SAND.



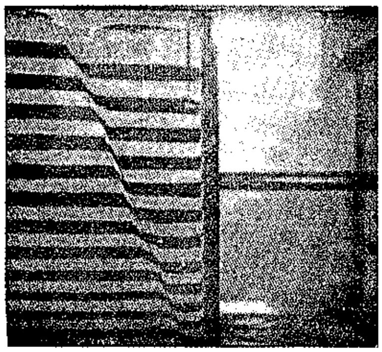
Wall Rotated 7½ Deg.



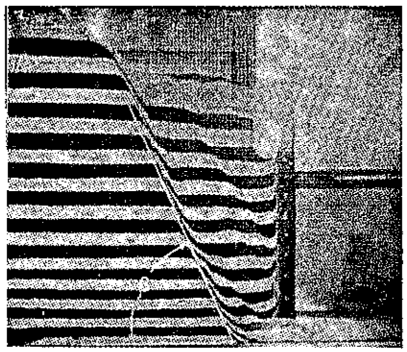
Wall Rotated 30 Deg.



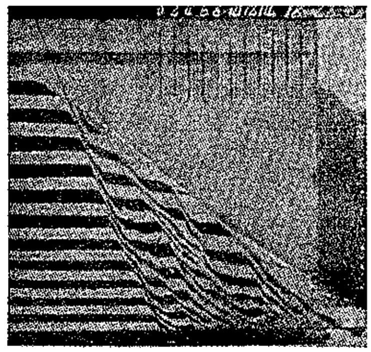
Wall Turned into Horizontal Position.



Wall Moved Forward 2 Cm.

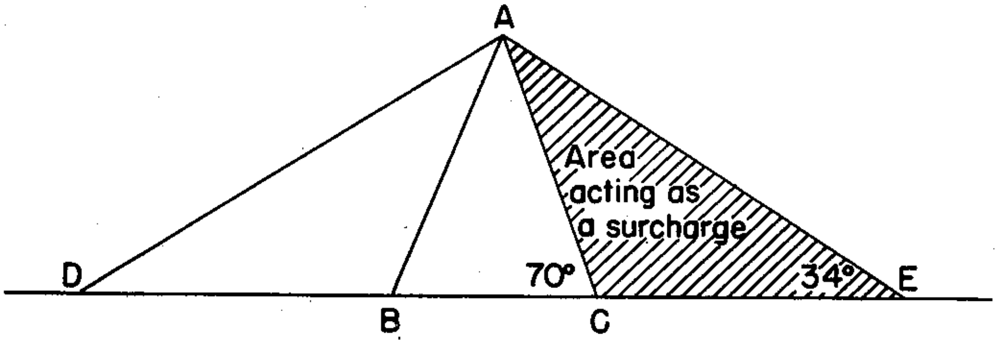


Wall Moved Forward 6 Cm.



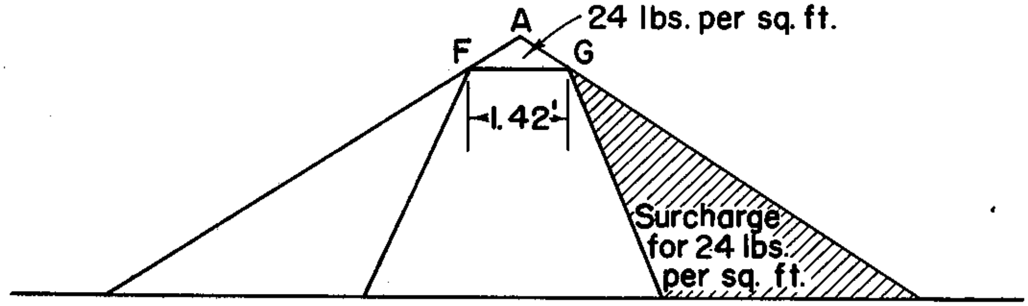
Wall Completely Removed.

Fig. 4



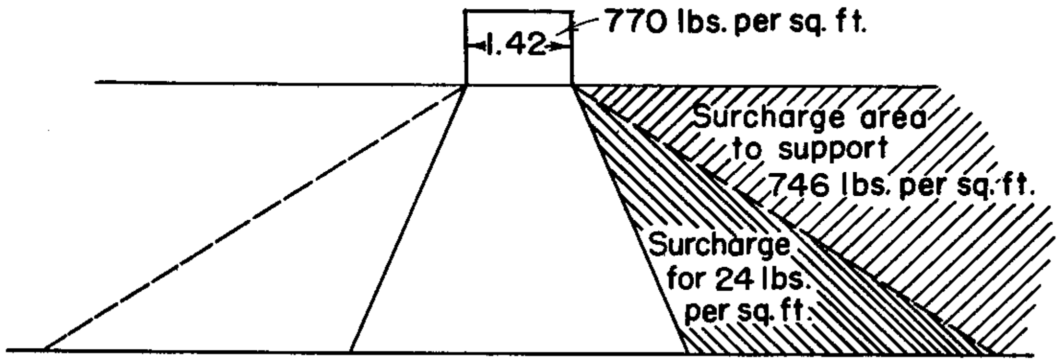
Equilibrium of a Dyke or Cone of Sand

Fig.5



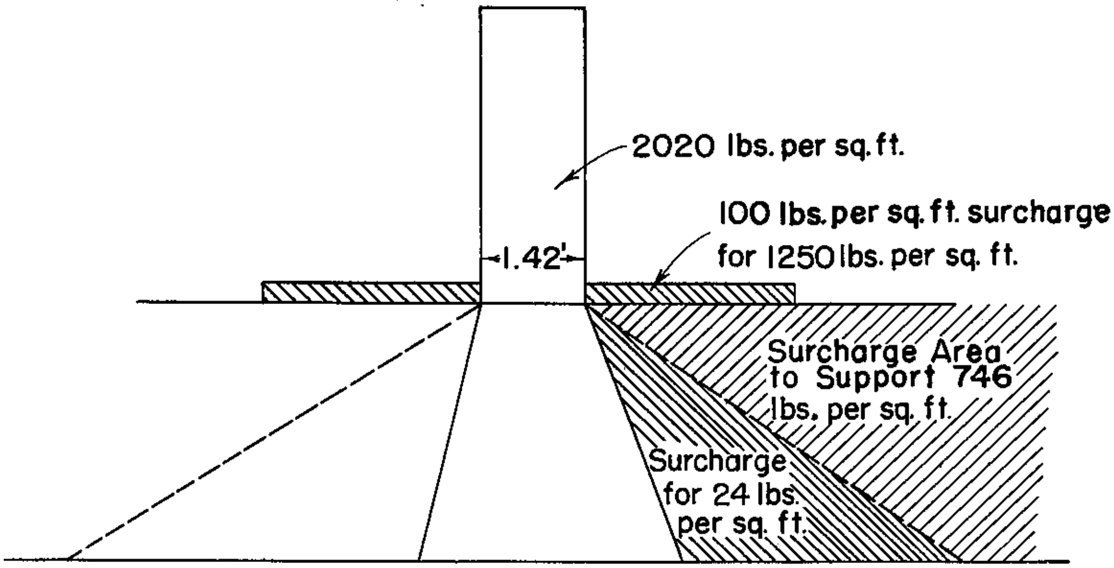
One sq. ft. Will Support 24 lbs. of Sand

Fig.6



One sq. ft. Will Support 770 lbs. of Sand

Fig. 7



One sq. ft. Will Support 2020 lbs. of Sand

Fig. 8

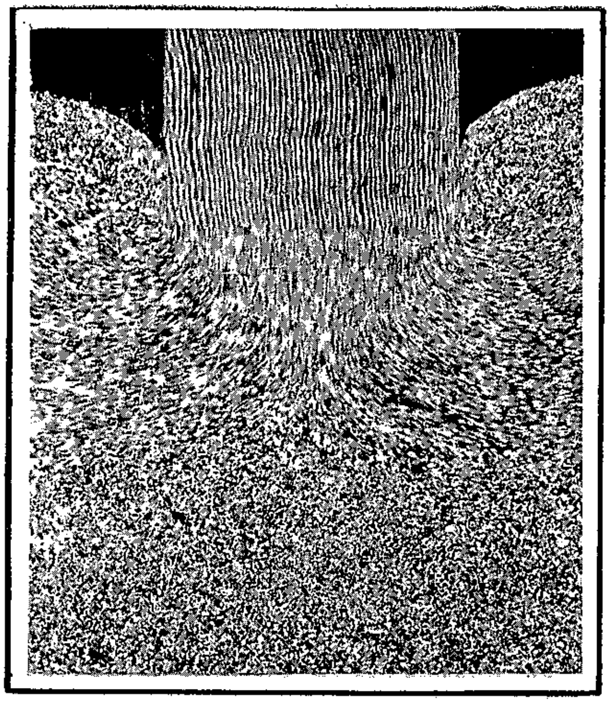


Fig. 9

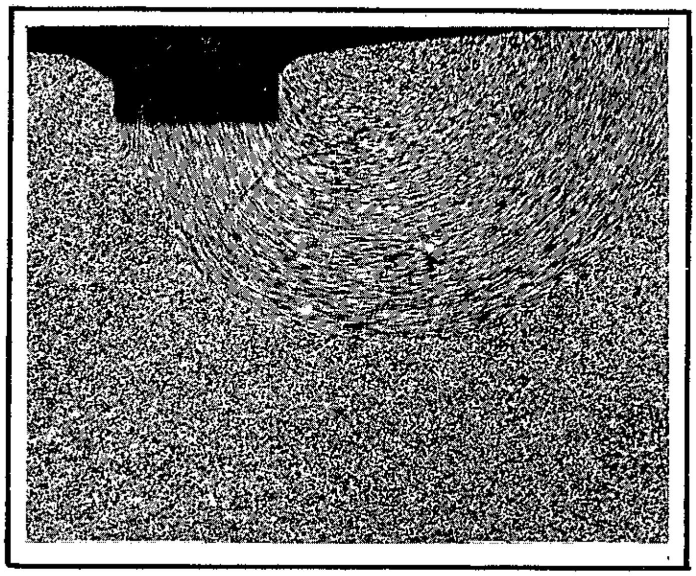


Fig. 10

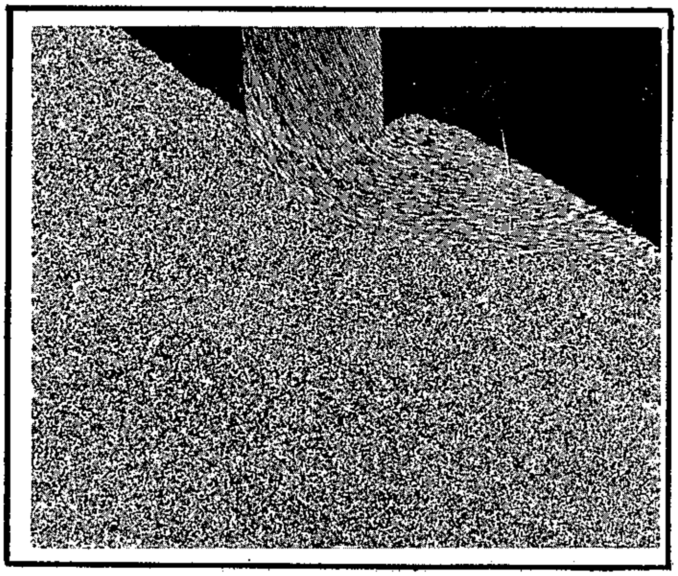


Fig. 11

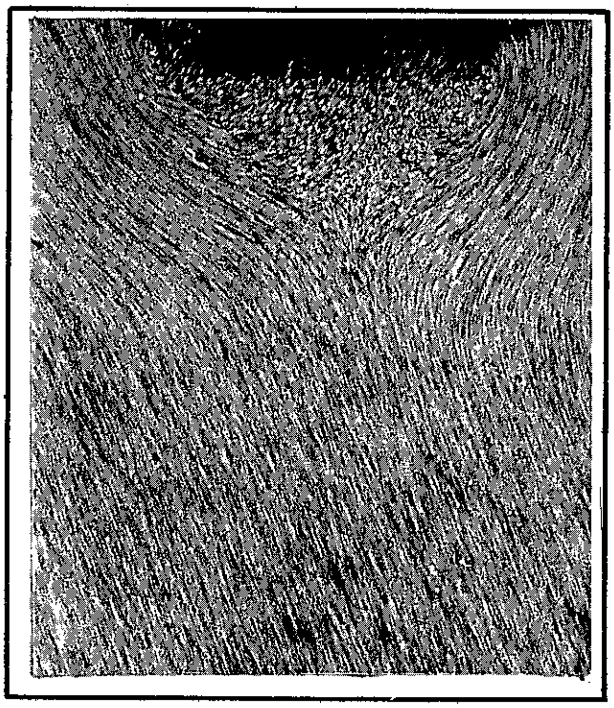
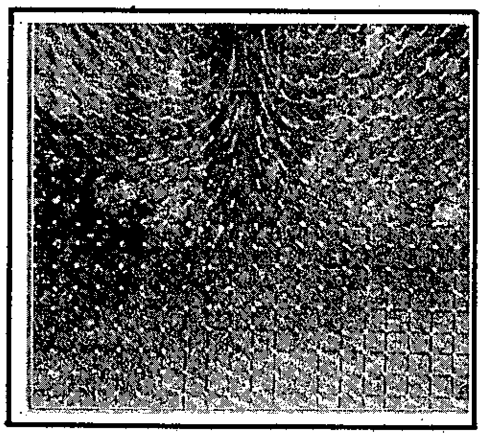


Fig. 12



Movement of Plastic Material under a Bearing Area

Fig. 13

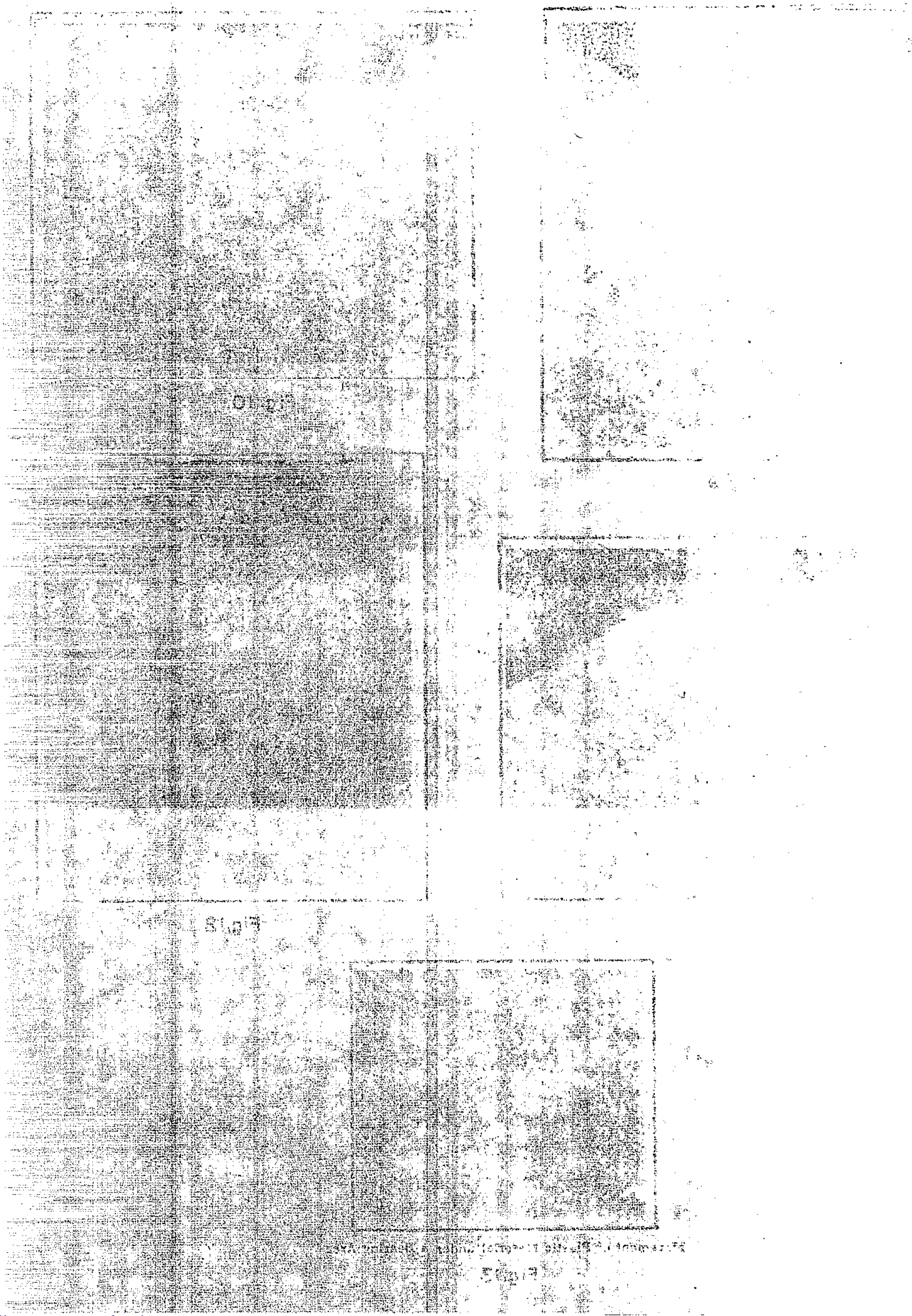


Figure 14 represents schematically the potential paths of individual particle movement at varying depths below the surface of the road. Figure 15 illustrates the probable lines of cleavage or fracture which will tend to develop in the less plastic mixtures if the movement is carried far enough. Figure 16 illustrates the distortion of hypothetical vertical lines and horizontal planes if the load is sufficient to cause deformation (this illustration shows that movement is not confined to a single "shear plane".) Figure 17 illustrates isobars of uniform pressure, commonly called pressure bulbs.

Thus, there are a number of phenomena which have been observed and studied, all of which are present and develop simultaneously when any extensive mass of granular material is deformed by pressure over a limited load area. Wheel loads on highway and airport pavements are examples of limited load areas on an extended plane surface and it is important that all of the most significant movements be observed in order to understand the mechanism involved.

The designing engineer is consciously or unconsciously seeking to provide conditions to maintain the status quo. A satisfactory paving surface is one which "stays put" retaining its surface contour and smoothness by resisting deformation and displacement, and therefore may be said to be in a state of equilibrium with its environment.

Most engineers recognize that loads carried by a pavement must also be carried by the subgrade and the underlying basement soil and it is taken for granted that the capacity of a soil to support a load is in turn greatly enhanced by the presence of a pavement and base between the load and the soil. To many engineers it has seemed to be an obvious conclusion that the superimposed layer of pavement and base is effective because it tends to "spread the load". Figure 18 is a sketch of a load distribution pattern as conceived by certain investigators. As will be noted from the photographs and sketches 9 to 17, there is little real evidence to sustain this concept of a "cone of distribution". While this assumption underlies many formulas proposed for computing pavement and base thickness, the lack of agreement and indifferent success with most of these formulas suggests that something may be wrong with the basic concept or premise.

Referring to sketches 14 to 17, inclusive, it is clear that if the load force exceeds the resistance between the soil particles located vertically below the load, the only path available for movement is in a lateral direction and therefore, any such tendency to move can be counteracted by adequate lateral resistance from material outside the load area. If the surrounding mass should yield, the path of least resistance is upward and is in turn opposed or balanced by the downward pressure (weight) of the superimposed layers, i.e., the base and pavement.

The special case of pavements, bases and subgrades.

In the simple case of a uniform subgrade soil covered by a pavement, we may consider two effects. First, if the paving slab possesses any flexural strength, it will undoubtedly tend to reduce the pressure on the subgrade through beam action or slab action which, of course, requires tensile strength. (In the case of bituminous pavement and granular bases, this is the minor effect). Second, a pavement (including any improved base) tends to restrain the upward movement of the soil beyond the area covered by the load. This restraint will depend upon the flexural strength of the pavement and pavement flexural strength will be more effective through restraining the upward movement of the subgrade than by reducing the downward pressure beneath the wheel. (This means that the most effective position for mesh reinforcement in a concrete pavement would be near the upper surface and not below the neutral axis.) The restraining action of the slab is further enhanced by the weight of the pavement, and this property is, of course, possessed by all pavements and by all layers of base or subbase so that with any type of base or surface the movement of any unit of the soil is restrained in direct proportion to the weight of the layers above regardless of type. However, the effectiveness of this restraint depends on the amount of friction in the subgrade material as only that portion of the resistance that is due to friction can be increased by pressure developed between the soil particles.

It is, or course, true that cohesion between the particles or tensile strength at any level below the surface will have some effect in restraining movement but the importance of tensile strength or cohesion in the pavement, base or subgrade is minor in the lower levels but increases in effectiveness as the upper surface is approached. Figure 21. Conversely, the effect of friction between the particles becomes increasingly important in the lower levels. (For example, the two properties could be segregated and efficiently arranged by placing a layer of granular material over a soil and covering the surface with a steel mesh mat.)

It would seem that several concepts are in need of some revision or the meaning of certain commonly-used terms should be clarified or amplified. These matters are important because when the use of a certain term or certain nomenclature invokes a mental image that is inadequate or "off the beam" the engineer is not likely to reach correct conclusions nor work out a sound design.

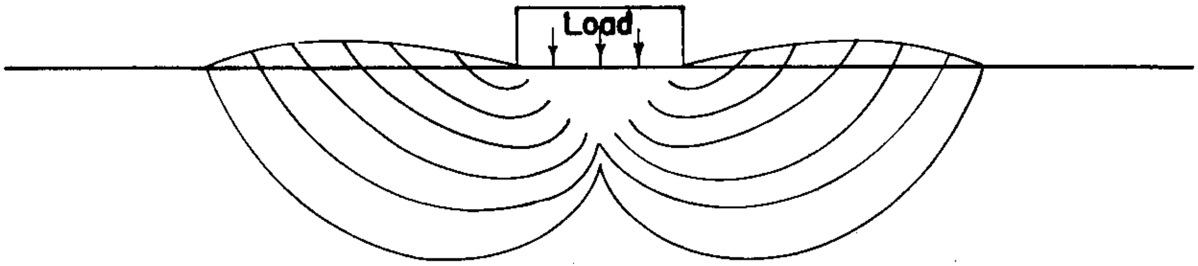


Fig. 14

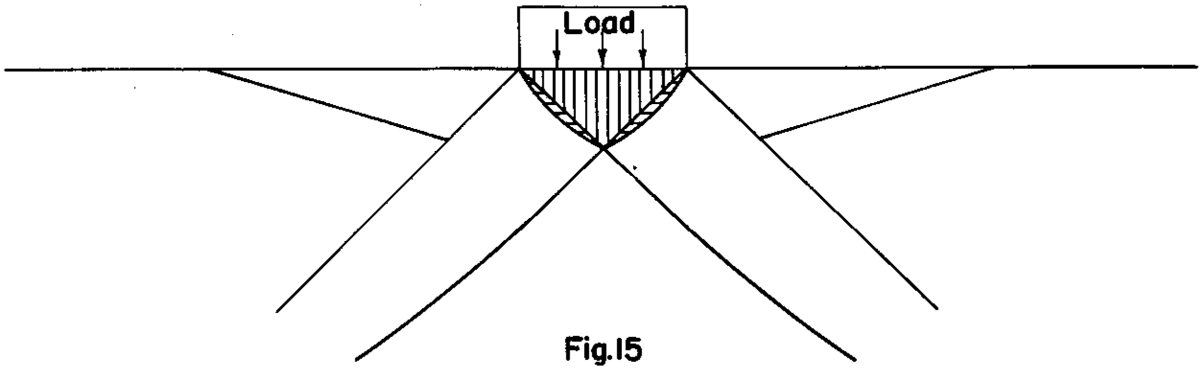


Fig. 15

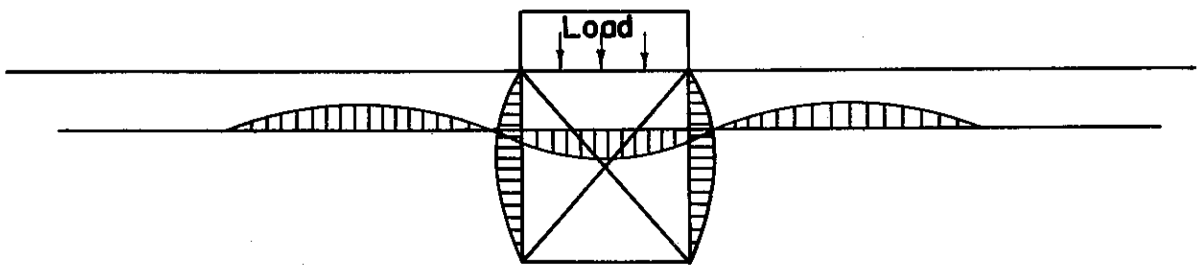


Fig. 16

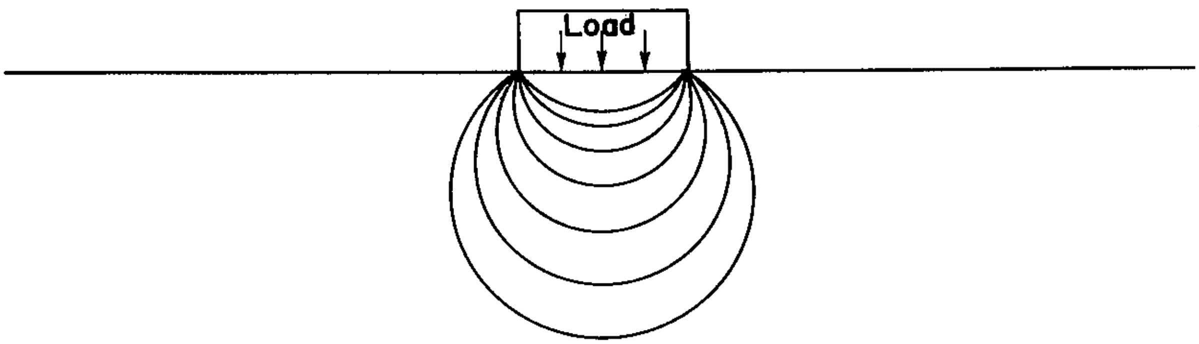
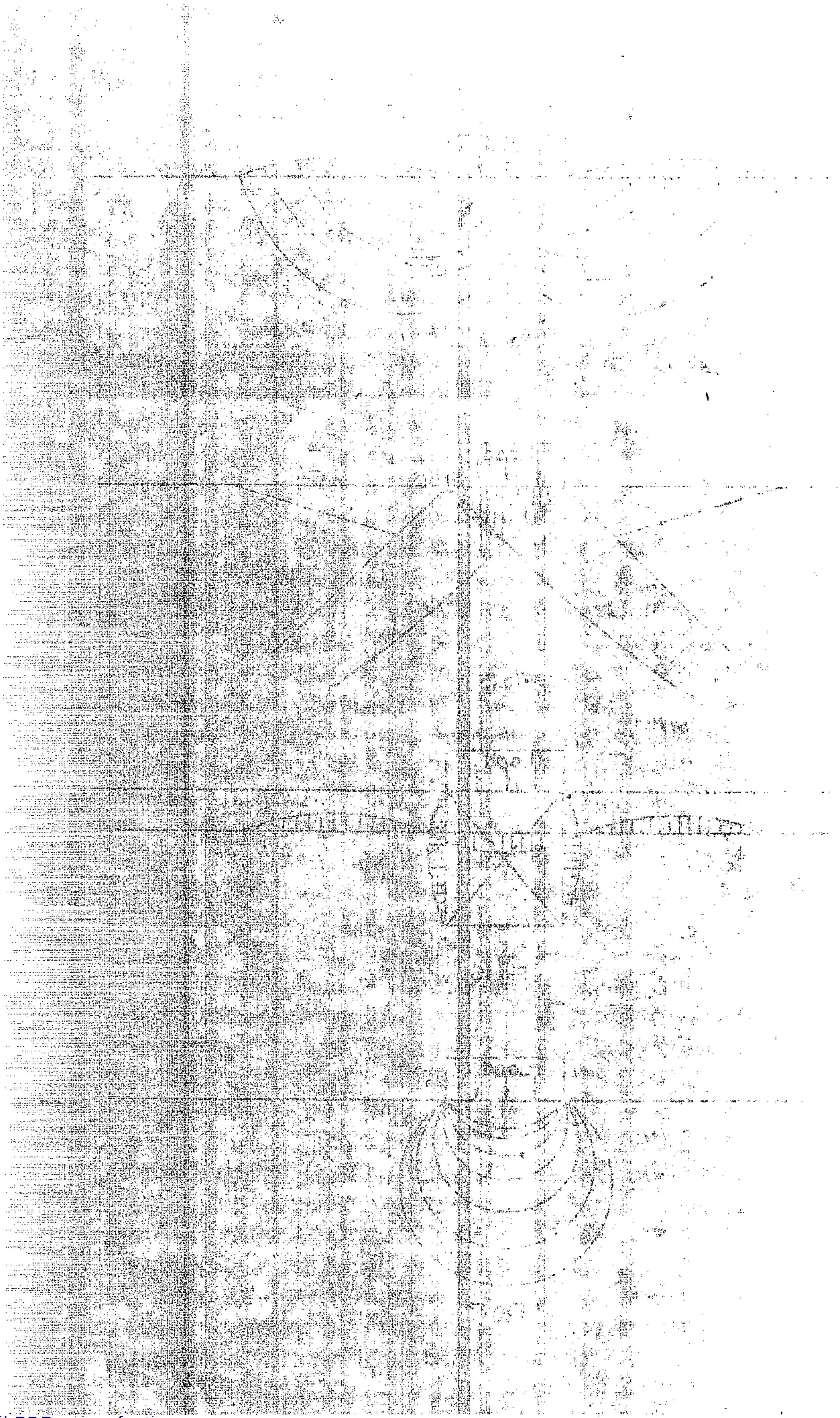


Fig. 17



In the author's opinion the word "failure" is one of these much overworked and oversimplified terms. It is probable that the breaking and crushing of a concrete cylinder is the most typical example of the concept of failure and having once failed, the structural integrity of a material or member of this type is forevermore vanished. The behavior of plastic materials or masses of granular particles is somewhat different. If a bed or a specimen of uncompacted clean sand is loaded to produce "failure", we have actually accomplished nothing more drastic than a more or less temporary change in the shape of the body. The mast of granular material has retained all of its original properties and is again ready to receive a load and will still offer as much resistance to movement as in the initial case. The same is true of plastic or over-rich asphaltic pavements. They can be classed as unsatisfactory because of their inability to retain a desired shape or surface contour. But to state without qualification that a test specimen of soil or bituminous mixture has shown "failure" is an oversimplification which is often very misleading.

It is furthermore true that virtually all materials show a different response to repeated light loadings as compared to a single load of sufficient magnitude to cause deformation or rupture. This is borne out by fatigue tests on concrete, metals, etc. It is reported that concrete will fail in flexure if constantly repeated stresses exceed 50% of the ultimate strength (7). Bituminous pavements may become distorted and unsatisfactory from a traffic standpoint through the accumulated distorting effects of a large number of traversing wheel loads even though each repetition of load produces an almost imperceptible movement. A difference however, lies in the fact that elastic substances are weakened while plastic materials often are not. Therefore, while the effects of load repetition may lead to "distress" in both cases, the mechanism of "failure" in plastic material is not necessarily identical with fatigue failures in a relatively rigid body.

It is furthermore apparent that the effects of the time element (that is whether long, continued versus repeated loadings) will show marked differences depending upon whether or not a viscous liquid is present in the supporting medium.

This would seem to be a good place to repeat some rather simple laws which describe the behavior of dry sliding friction between solid bodies and of liquid friction.

According to Sir W. B. Hardy in his paper entitled "Friction, Surface Energy, and Lubrication", (8) "two kinds of friction may be distinguished, the internal friction of fluids, usually called their viscosity, and the surface friction of solids which, in contrast with internal friction, might be called external friction.

External friction is the resistance to relative motion which two solid faces in contact offer; it is the reaction to the traction of the interface, and there are two kinds, kinetic and static, according as the traction does or does not cause slipping.---

External friction is subject to a law, formulated by Amontons in 1699, according to which the resistance to relative motion is independent of the area of the applied surfaces, and varies directly with the force, called the load, which presses them together.

Amontons' law may be put: For the same solids and the same lubricant, the tangential reaction (friction) per unit area is dependent only on the pressure."

In the description of liquid friction, Hardy quotes from works by Reynolds, Sommerfeld and Rayleigh. The following comments are based on Hardy's review.

When a viscous liquid is held between two plates separated by the liquid in question the resistance to sliding will be virtually independent of the pressure but will vary directly with the area and directly with the speed. This behavior bears a suspicious resemblance to the so-called "cohesion" effect that appears constantly in the literature on soil mechanics and is usually defined as that portion of the total resistance that is independent of the pressure. It appears that the nature of this cohesive resistance may be better understood if it is recognized as due to "the simple internal friction of a viscous fluid".

The laws which influence the movement of materials affected by external friction between solid particles and the internal friction of viscous liquids are in effect almost diametrically opposed.

A mass of rock particles, such as dry sand, crushed stone or gravel will perform in close agreement with the first law. Stabilometer tests indicate that changes in gradation within wide limits will have little effect on the frictional resistance as a whole. Practical corroboration is furnished by the easily made observation that stable foundations and bases have been constructed from aggregate gradings ranging

Illustrating the Misleading
Concept of "Load Distribution"

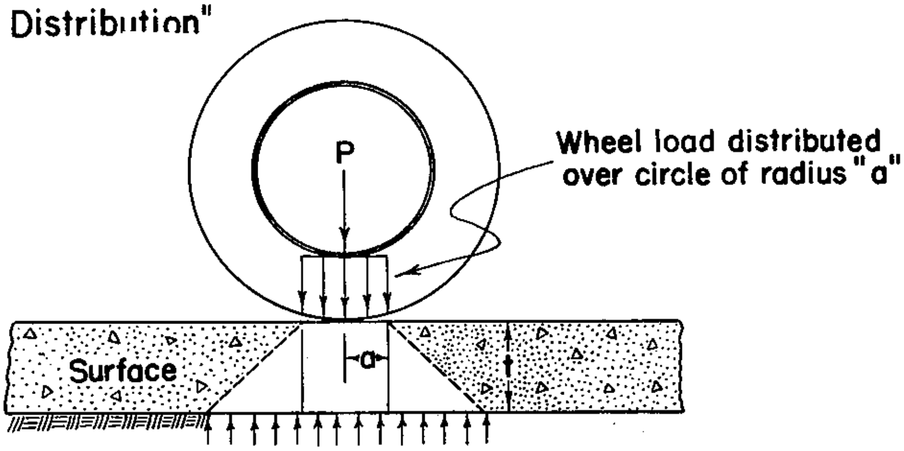


Fig.18

Direction of Forces Involved in
Stabilometer Test

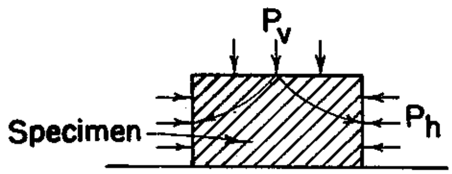


Fig.19

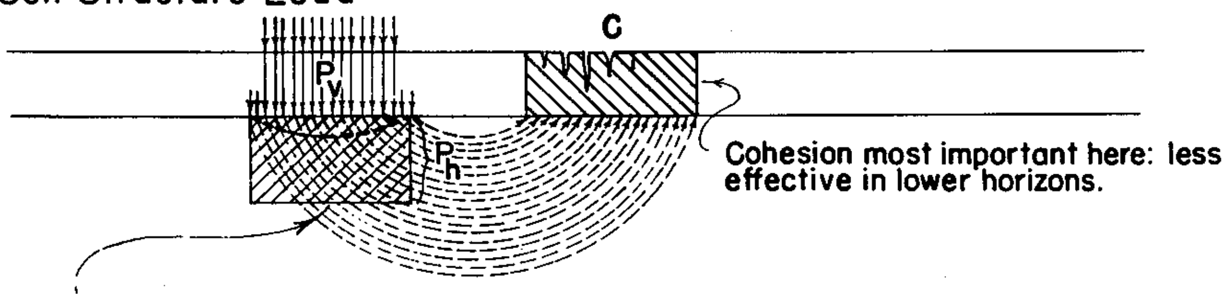
P_v = Vertical Pressure Applied

P_h = Horizontal Pressure Developed (Stabilometer Reading)

Direction of Forces Involved in
Cohesiometer Test

Fig.20

Direction of Principal Destructive
Forces Involved in Pavement and
Soil Structure Load



Friction most important here: less effective and less essential in upper layers.

Fig. 21

all the way from fine sand to coarse Macadam types with every sort of intermediate variation. Properly compacted and confined, any of these types can be made sufficiently stable. (The surface texture and shape of the particles will be far more important than the grading). Lubricated aggregate or soil mixtures, (for example, aggregates mixed with too much asphalt or with excess clay and water) present a case where the more or less rough stone particles are partially lubricated, although the sliding friction may not have been completely eliminated. Such a mixture will not follow either set of laws exactly. The greater the amount of lubricant present and the more effective the lubricant, the more nearly will the deformation characteristics of the material follow the stated law governing liquid friction, and as pressure has little effect on the resistance it naturally follows that wet lubricated clays show little response to the effect of surcharges; i.e., to increased pressure on the sliding planes, Figure 3. When particle friction is high and lubrication absent, any surcharge or any means for increasing pressure (compaction) between dry particles will increase the resistance to movement.

We thus have a variety of effects on stability when the quantity and nature of the lubricating liquid is changed because the cohesion effect in soils (which depends either on asphalt or water) will usually increase up to a point as a liquid is added to a soil mixture but the friction will usually decrease simultaneously due to lubrication and more resistance is lost through reduction in friction than is gained by increased cohesion.

In order to evaluate any engineering material adequately it is necessary to measure its significant properties, but adoption of appropriate tests for soils has been retarded by certain concepts. The errors are of two kinds, first is the habitual use of phraseology resting on assumptions. The term "bearing value" or "supporting power" is often applied directly to test results derived from certain types of testing equipment although the stress conditions developed in the test may be greatly different both in magnitude and dimension from those produced in the prototype.

The word "shear" and reference to shear tests appears frequently in all literature on soil mechanics. It is pertinent to point out that the term does not definitely imply any property of materials. "Shear" is a type of movement or stress which can be developed or applied to any material whether it be glass, steel, wood or concrete. Likewise, several other terms should be rescrutinized in order to recognize their actual meaning and proper place in the scheme. These terms might be classed as belonging to the geometry of testing materials rather than as defining specific qualities or properties.

Thus, we can apply compression loads, bending loads and shear to almost any substance. But each material resists the application and direction of these mechanical movements through the possession of simply innate properties which vary only in degree or magnitude between different materials. The ability of materials to resist stresses in tension is due to the forces of molecular attraction. (Presumably we do not pull molecules in two by mechanical forces.) The mechanics of granular masses is, however, on the whole more simple inasmuch as macroscopic discrete particles normally maintain their individual integrity, and molecular forces in soils may or may not influence particle friction although they undoubtedly are involved in the cohesion effect produced by liquid films over large surface areas.

Second, is the almost universal practice of testing specimens of soils which have been artificially prepared under compaction methods which are utterly unlike anything operating in nature or developed by construction equipment.

As pointed out above, all soils and mineral aggregates are collections of particles and the mechanics of soil deals with the actual or potential movement between the individual fragments. Hence, the conditions existing at the points of contact between the particles is of paramount importance.

The only conceivable properties by which a mass of granular materials may resist movement or deformation are:

1. The resistance to relative motion which two or more solid particles in contact will offer. This resistance may be more or less accurately defined as friction. The total resistance will vary with the roughness of the particle surface and with the pressure which forces the particles into contact.

2. Cohesion or tensile strength i.e. resistance to stretching or pulling apart displayed by soil mixtures containing either hydrated colloids and clays or bituminous binders. The behavior is closely influenced by the laws of liquid friction.

3. Inertia. (Possessed by any mass and ordinarily need not be differentiated between pavement or bases of different materials).

There are no others: and only when these properties are measured with reasonable accuracy and their interrelation understood will it become possible to predict the behavior of soils and bituminous mixtures under a variety of loading conditions. Soil test specimens may be subjected to compression, tension, bending or shear stresses, but no other property will be represented in the test results except uncertain proportions of friction and cohesion as defined.

Test data derived from simple shear tests, extrusion tests load-penetration tests and unconfined compression tests will all reflect some arbitrary composite of the two elements, friction and cohesion, and it is ordinarily impossible to determine the relative proportions of each which make up and are concealed in the laboratory test results.

In view of the fact that these two elements respond differently or combine variously (depending upon conditions of dimension, loading, time and temperature) to produce the sum total resistance of a bituminous pavement or subgrade, it is therefore, impossible to establish a consistent or even parallel relationship between such composite test data on small specimens and the performance of the prototype.

Hence, separate tests should be employed which indicates: First, the internal friction or sliding resistance which exists under the appropriate worst conditions of load, moisture, compaction and temperature that are expected to exist in service. Second, the cohesion or tensile strength under similar typical conditions should be evaluated. The two quantities must then be related or combined in proper proportion to accord with each special case or circumstance and their ratios will change depending on the load area, duration of loading, depth below the surface, or any variation in the geometry of the actual service condition.

A test reflecting internal friction may be made in the stabilometer, (Figure 19, 31 and Figure I in Appendix I.)

Cohesion of the liquid films or tensile strength of the mass can be evaluated by means of any apparatus which measures the force required to pull a test specimen apart, Figure 20. (For example, the Cohesiometer).

The properties reflected by these two measurements may be visualized as "dominating" certain limited regions of the combined structure of pavement, base and subgrade as shown in Figure 21.

The cohesion is chiefly important through adding to the pressure normal to the planes of sliding and thereby magnifying the resistance due to friction.

The diagram in Figure 21 is an attempt to visualize the conditions indicated under problem two in Figure 1 where it is shown that there are three major factors influencing the ability to support loads.

Using the symbols shown on Chart, Figure 1, the factors have the following general relationshipP

$$T = \frac{KD\ (90-R)}{S}$$

Where T* = thickness of all layers, including pavement, base, subbase, etc., above soil in question, inches.

K = a constant: numerical value depending upon the units used and upon the factor of safety desired

D = deforming effect of pneumatic tired wheel loads

R = resistance value of the soil (range 0 to 100)

S = tensile strength of pavement or base or both

While the foregoing expression indicates the mathematical relationship between the several component parts of problem two, the formula, of course, is useless until some typical units and values can be assigned to develop an expression which can be used in the calculation of pavement thickness required to sustain a given magnitude of traffic.

*In the above and subsequent formulas T may mean the thickness of base and pavement when the quality of the subbase material is represented by R or T may be used to calculate the thickness of pavement or surface course when the quality of a granular base is being considered. In other words the thickness of any one or more layers may be calculated by the method indicated.

II

MATHEMATICAL RELATIONSHIPS BETWEEN MAGNITUDE OF LOAD, AREA OF CONTACT, LOAD REPETITION AND STRENGTH OF PAVEMENT AND RESISTANCE VALUE OF SOIL

By dividing the total deformation of a pavement surface showing grooving or rutting by the number of axle load repetitions required to produce the "failure" it has been found that a moving wheel load caused an average permanent deformation at each application of approximately 0.1% of the individual deflection noted at each passing trip.* This shows that the deflection, under each load application even in an inadequately designed highway, over a plastic soil, is approximately 99.9% elastic. In spite of this preponderance of elastic phenomena however, there is no doubt that the eventual failure is due to the accumulation of the small plastic movements. While the deformation is typical of a plastic material it appears that the stresses which cause these plastic strains might be analyzed from elastic theory.

Under a moving wheel load the resultant flow of a plastic soil material is most pronounced in a direction at right angles to the direction of traffic. The stress pattern involved in this flow might be approximated by means of the theoretical solution to either a circular load, a rectangular load or a strip load upon a semi-infinite solid. There are supposedly rigorous solutions to all three. The choice is a matter of opinion. The pattern of the stresses and most probable planes of slip will be different for the different types of loading. However, the inadequacy of the numerous formulas derived from elastic theory and from which design is based upon a calculated value of stress or strain at some predetermined point in the soil mass seems to indicate the futility of trying to reach a workable solution by this method alone. Therefore, we have used the theoretical treatment only as an aid to establish proper test procedure and to rationalize some of the principles brought out by experimental data and to obtain a better understanding of their limitations.

Let us consider the theoretical treatment of strip loading. The deformation of a soil mass caused by a repeated strip loading would result in a groove similar to that made by a moving tire. With each trip of the tire, the surface receives a certain unit load over the same area as with an application of the strip loading.

*Stockton Test Track and Brighton Test Track

Figure 22 represents a cross section under a strip load and shows the direction of the major principal stresses. Figures 23 to 26 show the direction of the most probable planes of slip when different degrees of particle friction and cohesion are present* Figure 24 shows the case of a soil having a low ratio of cohesion to friction. Figure 25 represents an intermediate ratio and Figure 26 a very high ratio. These throw light on the tendency of a cohesionless material such as sand to become compacted at the lower levels and also indicate the advantage of protecting granular base types with an upper layer having some tangible tensile strength. Figure 23 shows the effect of the weight of the soil on the direction of the slip planes. In this case the direction of the most probable shearing plane is less favorable, nevertheless the soil weight increases the pressure and hence, the resistance due to friction along these planes.

The ability of the loaded soil to sustain loads will depend upon the ratio of stress to ultimate resistance $\frac{S_\alpha}{C + p_n \tan\phi}$ along these surfaces most likely to slip.

Table I, Figure 27, shows the ratio of shearing stress to normal pressure, on the most probable planes of failure, at various points of the loaded soil. The failure planes are calculated assuming no cohesion. From this it is evident that except for a small volume of soil directly under the load, with no cohesion, gross failure should take place in a certain region even with a friction coefficient of unity. When loads are applied over larger areas (with constant pressure) the relative proportion of overstressed soil decreases due to the fact that the weight of the soil (especially beyond the load area) has a stabilizing effect; but, within the range of highway and airport loads, added resistance is necessary either from cohesion (tensile resistance) in the soil itself or from a cohesive surface which has the effect of providing a surcharge outside the load area before the amount of the upthrust becomes serious.

REQUIRED THICKNESS OF BASE AND SURFACE IS PROPORTIONAL TO THE WIDTH OF THE LOADED AREA

Although the picture becomes somewhat clearer from these theoretical studies, any attempt to use them quantitatively for design is certainly not warranted. As a basis for comparative considerations, the prospect is brighter.

*The method of calculating these most probable planes of slip is given in Appendix II.

DIRECTIONS OF MAJOR PRINCIPAL PRESSURES UNDER STRIP LOADING

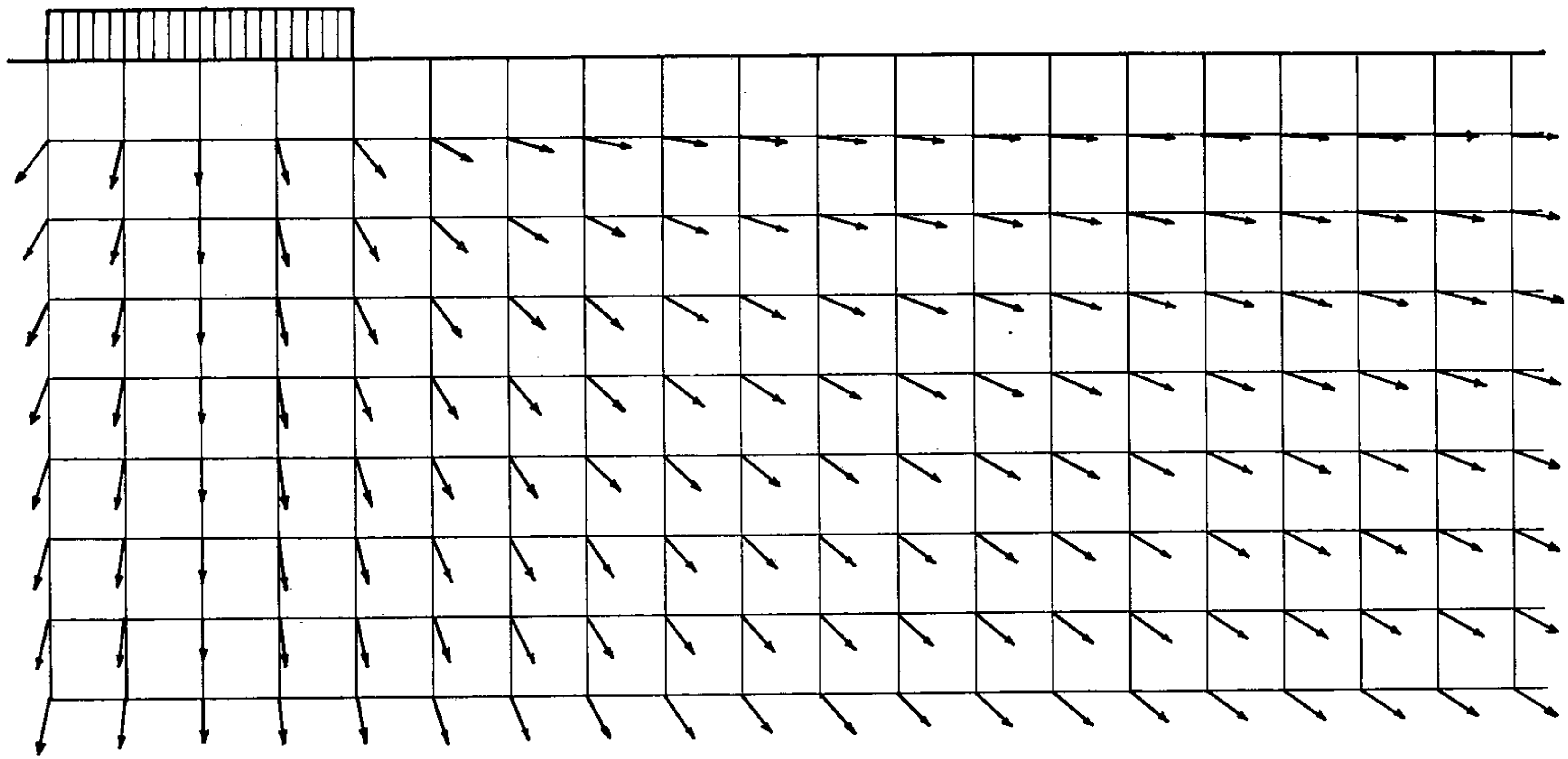
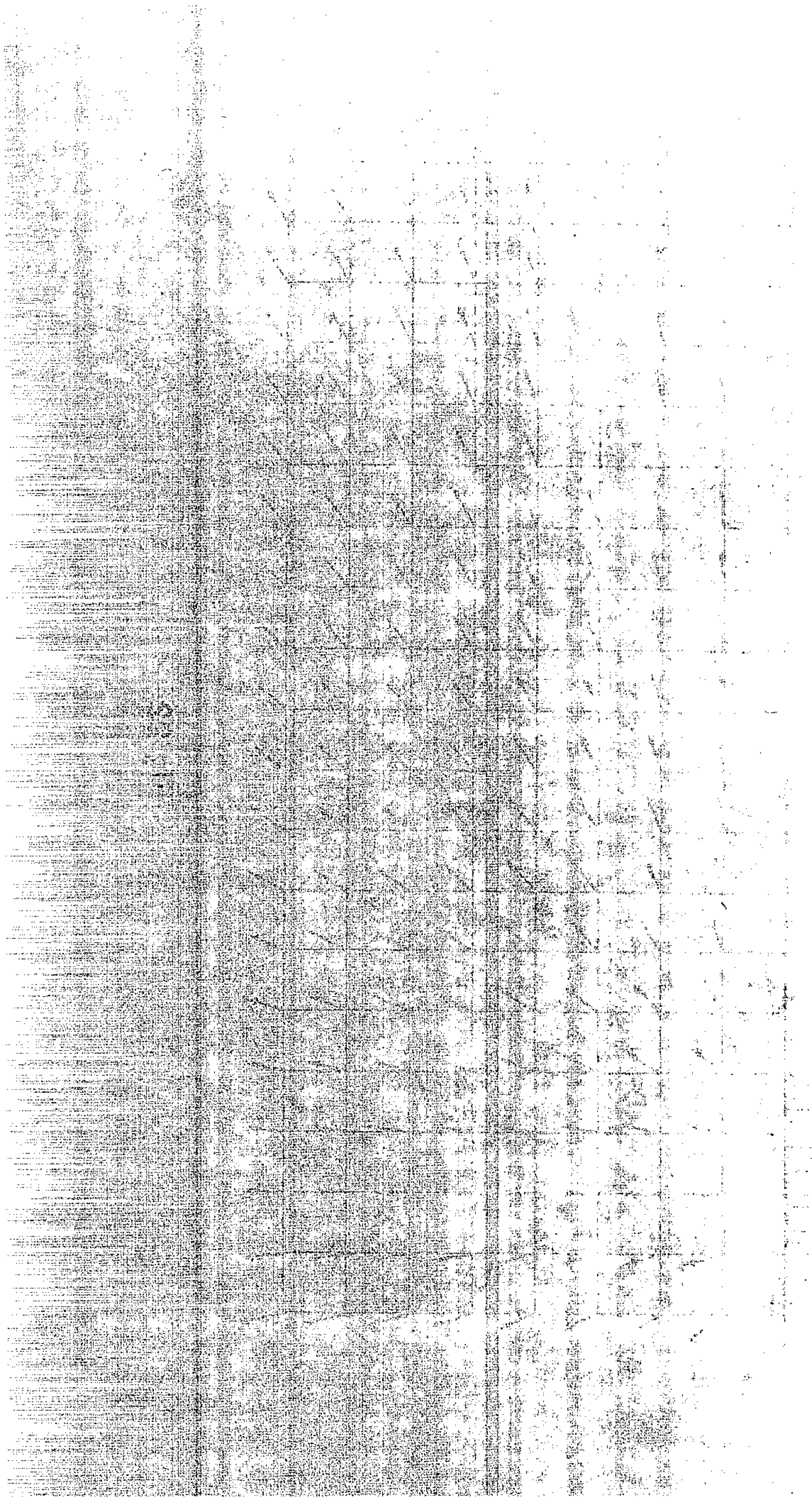


Fig. 22



PATTERN OF MOST PROBABLE SLIP SURFACES

$\frac{\pi c}{q \tan\phi} = 0$ Load = 6000 lbs. Wt. Earth = 120 lbs./cu. ft.

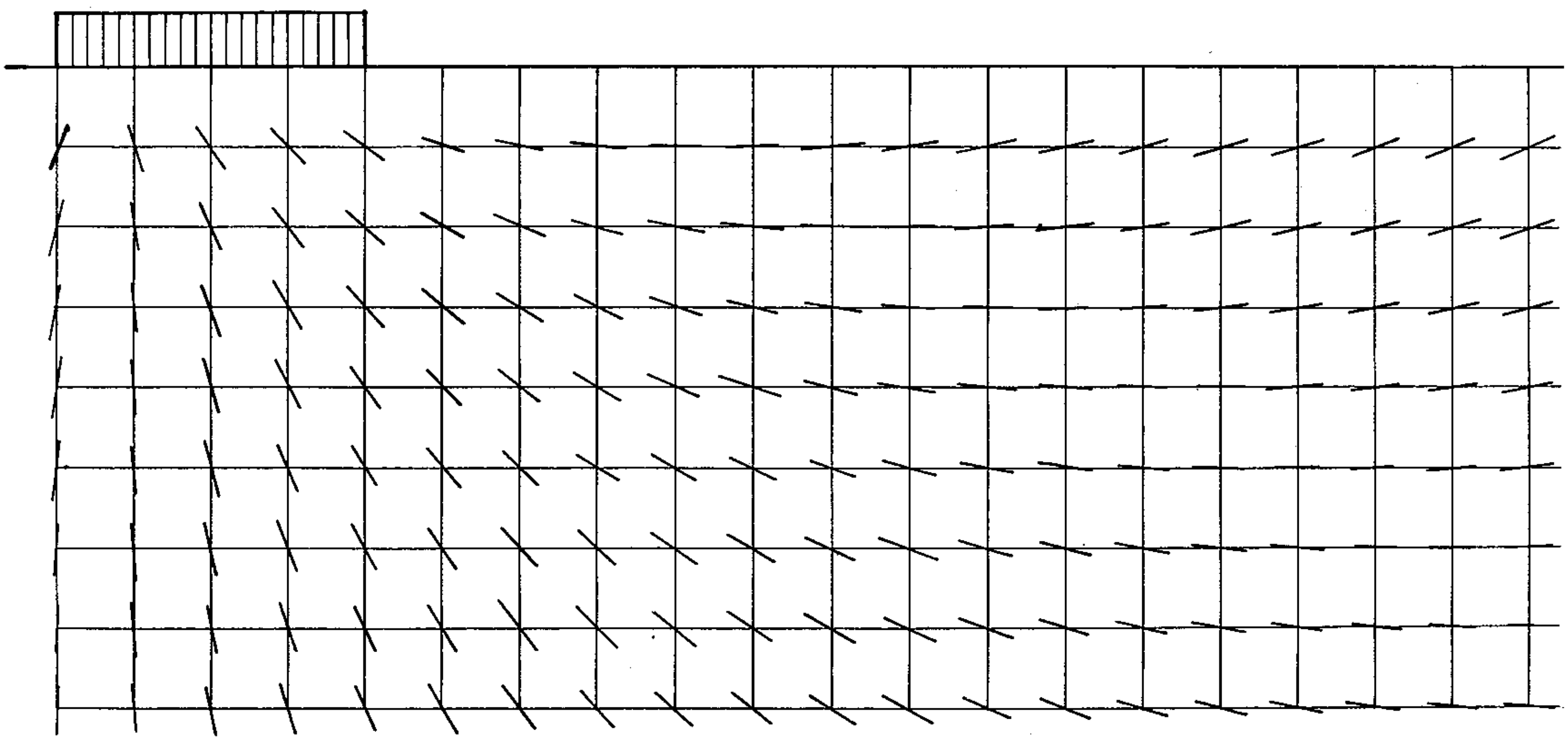


Fig. 23

PATTERN OF MOST PROBABLE SLIP SURFACES

$\frac{\pi c}{q \tan \phi} = 0$ Load = 6000 lbs. Wt. Earth Neglected

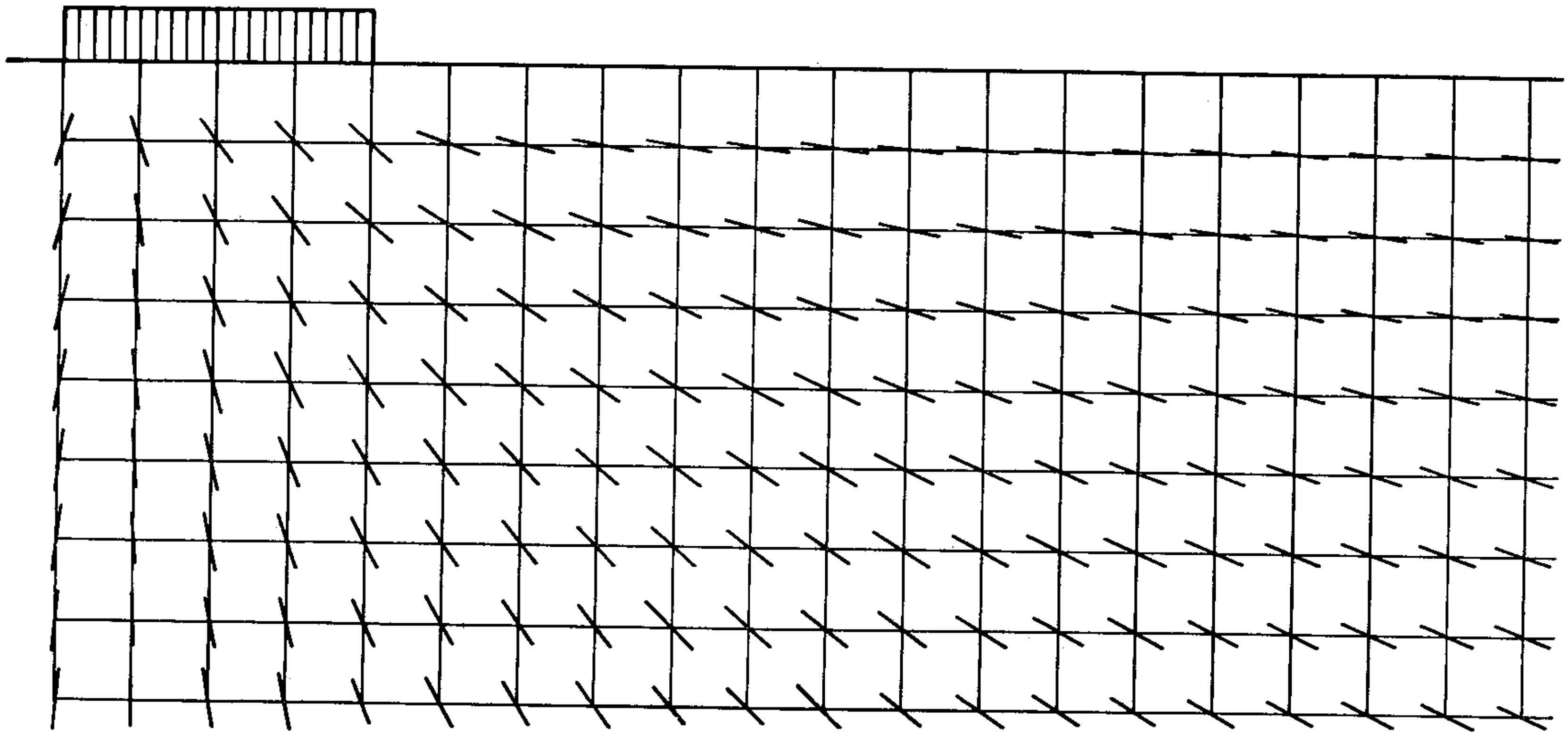


Fig.24

PATTERN OF MOST PROBABLE SLIP SURFACES

$\frac{\pi' c}{q \tan \phi} = 0.5$ Load = 6000 lbs. Wt. Earth Neglected

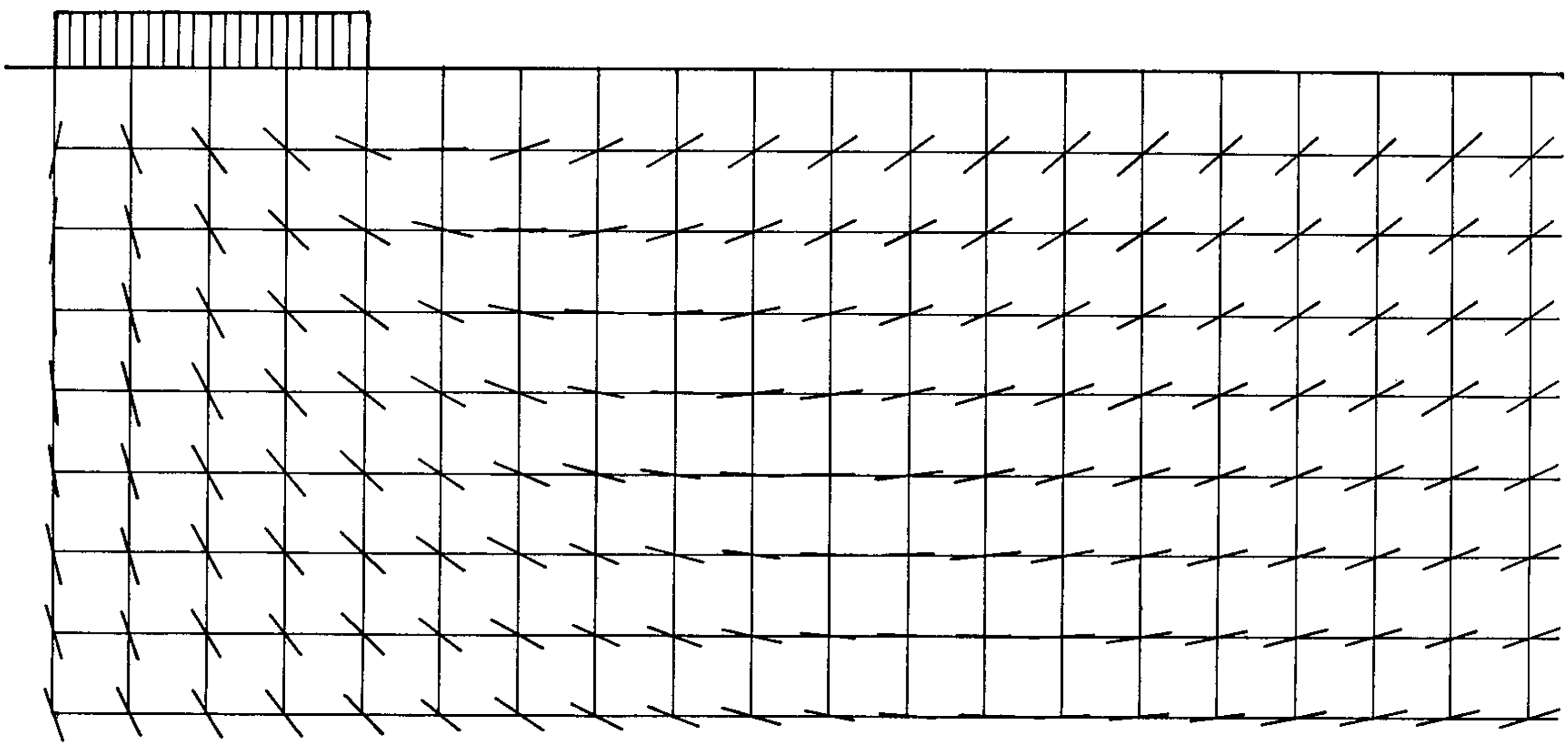


Fig.25

PATTERN OF MOST PROBABLE SLIP SURFACES

$\frac{\pi c}{q \tan \phi} = \infty$ Load = 6000 lbs. Wt. Earth Neglected

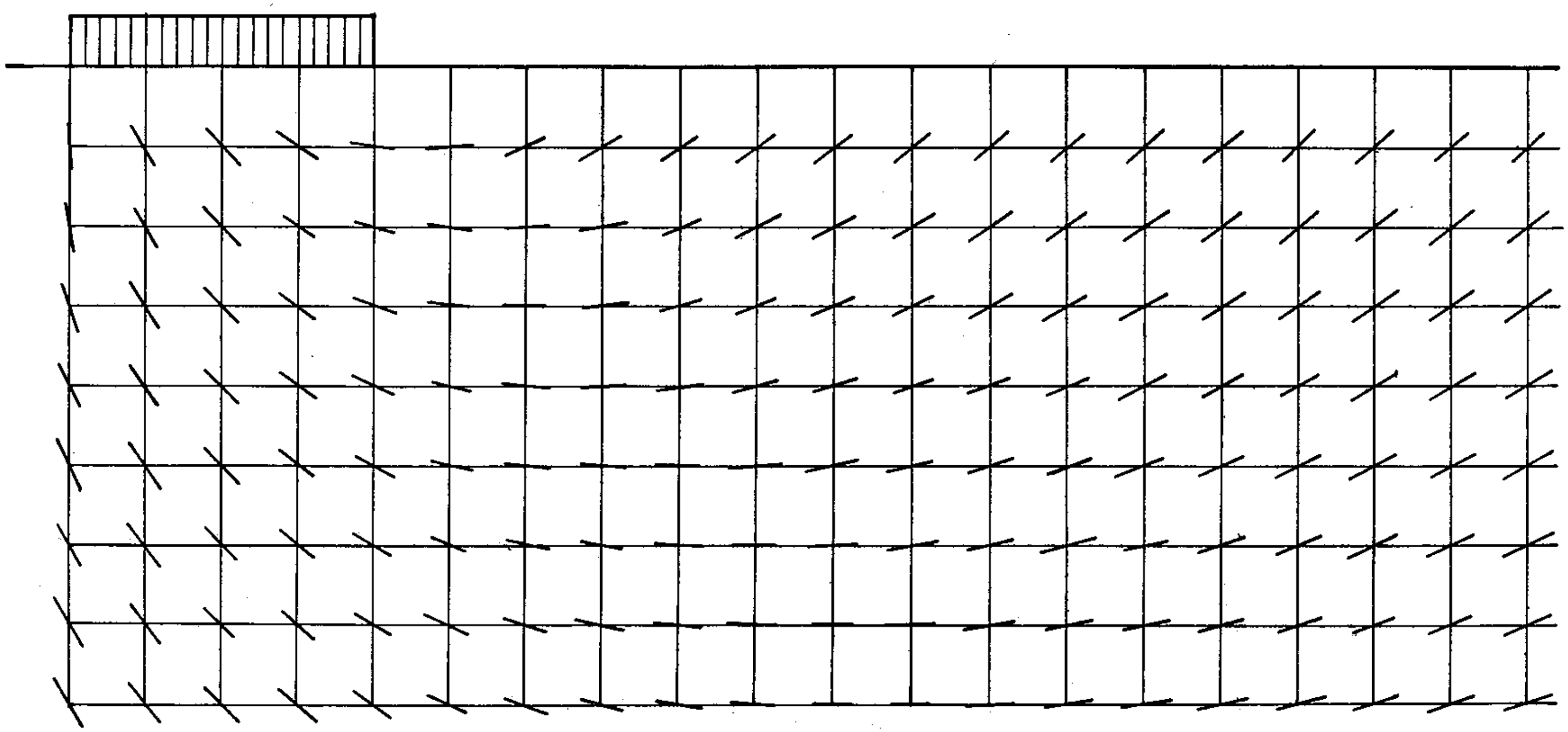


Fig. 26

SHEARING STRESSES (TOP) AND NORMAL STRESSES (BOTTOM) ALONG THE MOST PROBABLE FAILURE PLANES

Table I

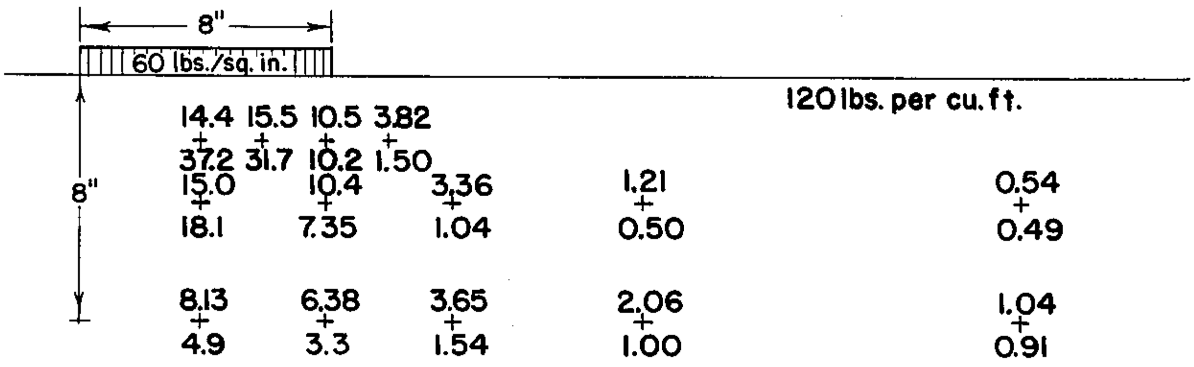


Table II

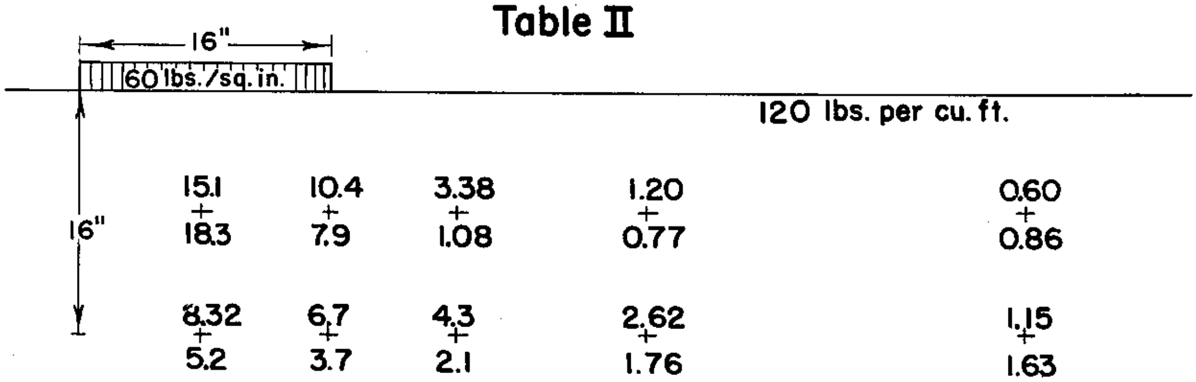


Fig. 27

Neglecting the weight of the soil, the formulas for stresses and strains and for direction of the most probable planes of slip are linear functions of the width of the load area whether it be circular, rectangular or a strip loading. The added complication of the weight of the soil affects not only the shearing and normal stresses, but also the angle of the most probable plane of slip. However, by a comparison of Figures 23 and 24 it appears that we can justify neglecting the effect of the soil weight on the direction of the probable planes of slip when dealing with the variations within the range of highway loads.

Table II, Figure 27, shows the changes in the shearing and normal stresses along the probable failure planes when the unit load is spread over an area having twice the lineal dimension shown in Table I. The effect on the stress pattern is the same as though the weight of the soil had been doubled. This difference due to the soil weight affects the frictional resistance only as it is only the frictional resistance that can be increased with an increase of load on the planes of sliding. In the vicinity of the wheel load the effect is small as the stresses produced by the soil weight are small compared to those produced by the load. Following the pattern outward, away from beneath the load, the stresses from the load become less and those from the soil weight proportionally greater; but the ratio of shearing stress to normal pressure becomes greater and it is evident that should there be appreciable tendency toward movement outside the loaded area most of the resistance to the movement must come either from cohesion in the soil, which is not affected by the soil weight, or must come from a cohesive surface mat resisting upward thrust which influence also is not affected by the soil weight.

From these circumstances it seems a reasonable assumption that for highway and even airport loads we may neglect the error caused by the weight of the different masses of soil involved when comparing the effects of loads over different sized areas and we may, therefore, state with some assurance that, providing the tire pressures are approximately constant and the tire imprints are approximately the same shape, and other things being similar, the necessary thickness of base and surface for the same volume of traffic will be proportional to the tire widths or the square roots of the tire contact areas. (Tire contact areas are more readily calculated from data on total loads and tire pressures than are tire print widths).

Some verification of this premise is shown in Fig. 28, which shows the relation between the number of load repetitions and the thickness of base and surface at which failure became evident for three different wheel loads. The slopes of the heavy lines are approximately proportional to the square roots of the loads. Actually they are proportional to the product of tire pressure, (which is slightly different for the three loads), and the square root of the tire imprint area, which will be explained later. It will be evident that the expressions $\sqrt{load}$ and $\sqrt{area}$ are parallel values when pneumatic tires carrying identical pressures are involved. It should also be noted that it is erroneous to assume that a tire print load is adequately represented by a circle of equal area.

The data for the 40,000 pound load and the 25,000 pound load are taken from a report on the Stockton Test Track constructed by the Corps of Engineers and for the 6,000 pound load from the Brighton Test Track constructed by the California Division of Highways.

REQUIRED THICKNESS OF BASE AND SURFACE IS PROPORTIONAL TO THE AVERAGE TIRE CONTACT PRESSURE

On page 19 it was established that with tire pressure and other things equal, the thickness of base and surface required varied as the square root of the tire contact area. The relation between the thickness required and the contact pressure when the area is constant is not so well established; however, comparisons between the effects of single tired loads and dual tired loads indicate that the required thickness of base and surface is directly proportional to the tire contact pressure. Also results of deflection measurements made a Stockton Test Track show that the deflections caused by the various loads (when measured at thicknesses of base and surface proportional to the square root of the tire area) are approximately proportional to the thickness of base and surface required to carry these same loads. Theoretically and experimentally, deflections, when measured at thicknesses of base and surface proportional to the square root of the tire area, are proportional to the product of the tire contact pressure and the square root of the contact area.

These facts suggested that the required thickness of base and surface would be proportional to the tire pressure. Figure 29 shows the theoretical rate of failure, (based on this assumption for the severity of the load), compared to the actual rate observed on the test track.

Points on the curve were calculated by means on the formula, page 24, after reducing all load repetitions to an equivalent number of 5000# wheel loads.

REQUIRED THICKNESS OF BASE AND SURFACE IS PROPORTIONAL TO THE LOGARITHM OF THE LOAD REPETITIONS

The relation between volume of traffic and thickness of base and surface has been determined by the interpretation of

RELATIONS BETWEEN LOAD REPETITIONS, WHEEL LOAD & REQUIRED THICKNESS OF BASE & SURFACE

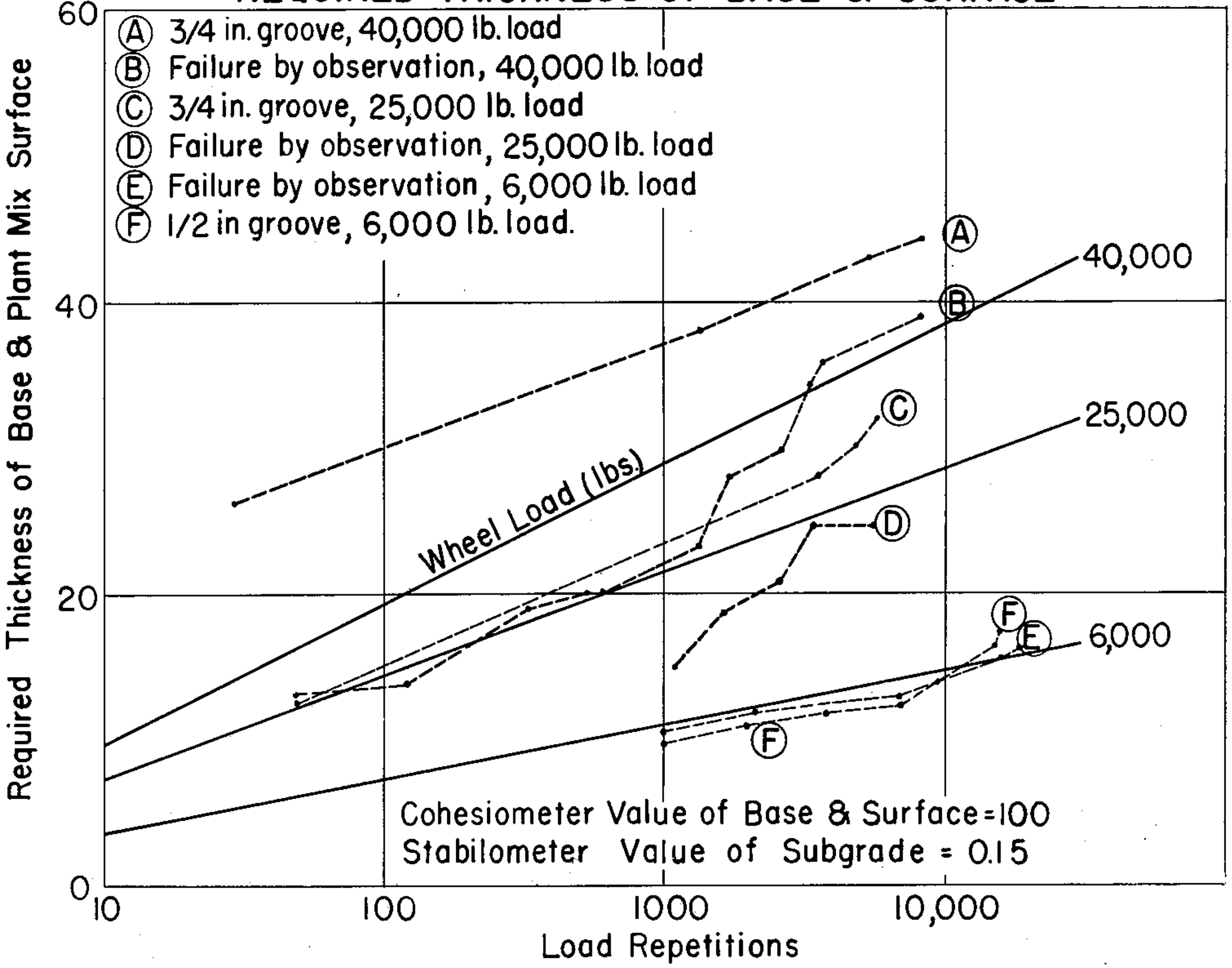


Fig.28

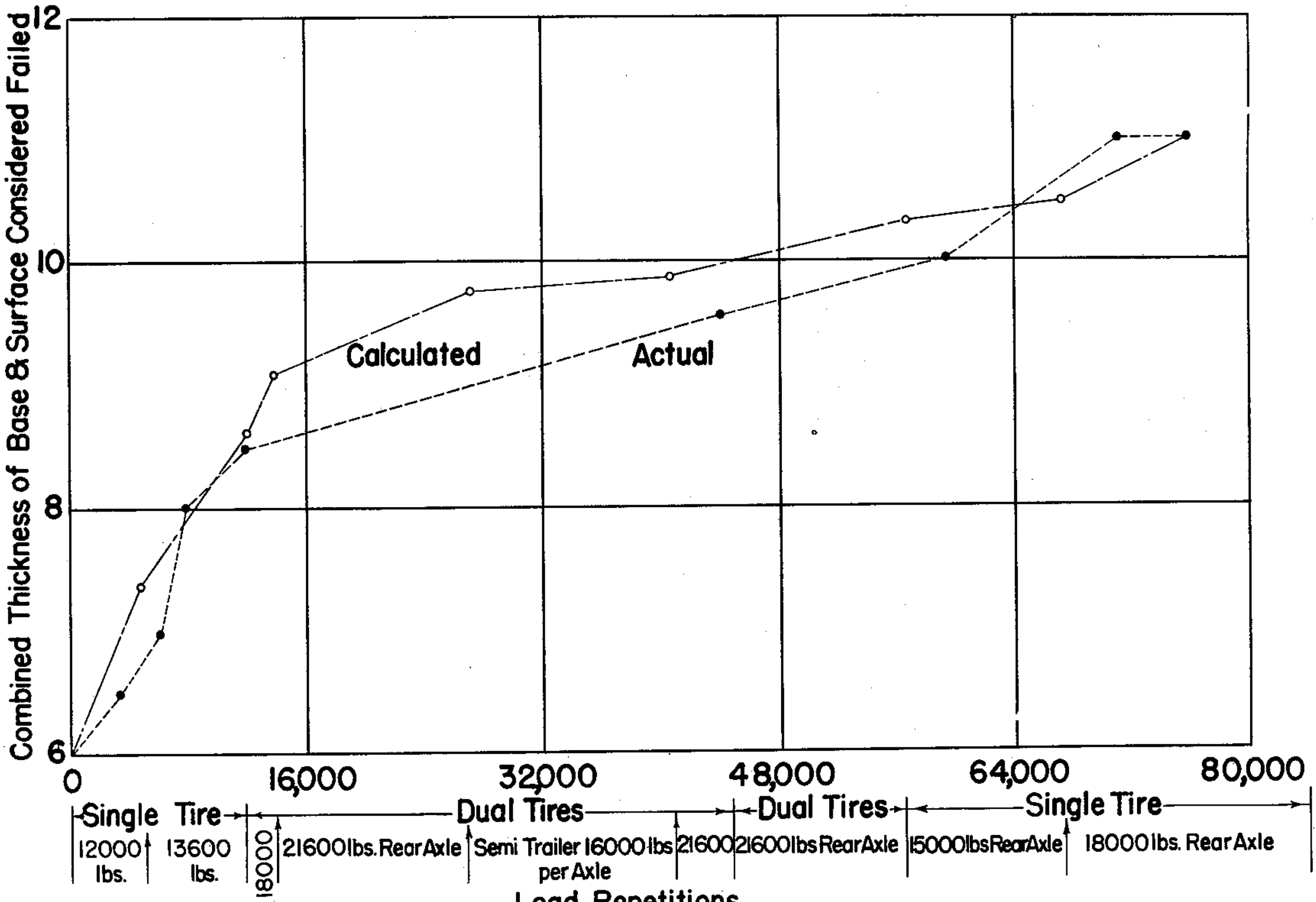
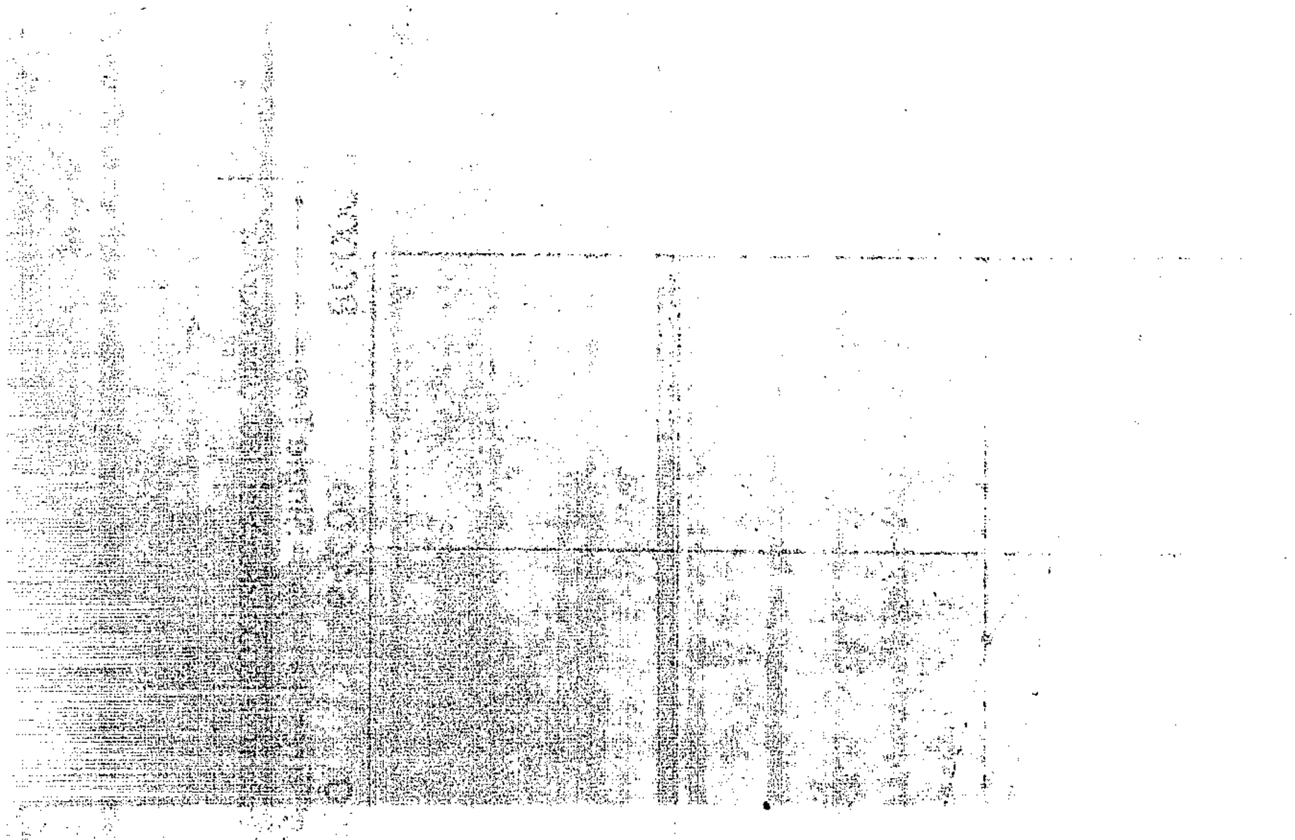
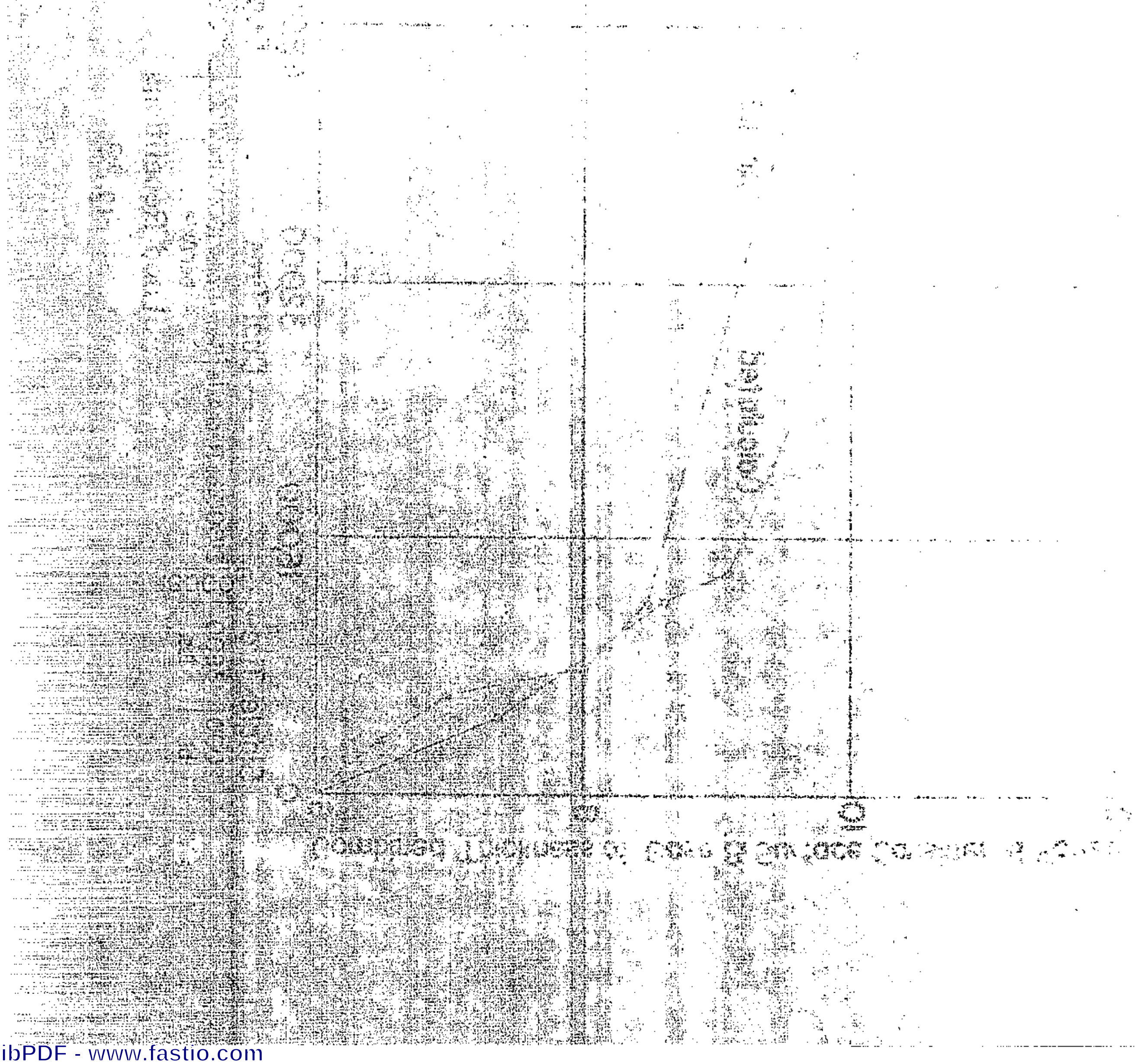

Combined Thickness of Base & Surface Considered Failed
12
10
8
6
0
16,000
32,000
48,000
64,000
80,000
Calculated
Actual
Single Tire
Dual Tires
Dual Tires
Single Tire
12000 lbs.
13600 lbs.
18000
21600 lbs. Rear Axle
Semi Trailer 16000 lbs per Axle
21600
21600 lbs Rear Axle
15000 lbs Rear Axle
18000 lbs. Rear Axle
Load Repetitions

Fig. 29





experimental data obtained from the test tracks. The data shows that to prevent plastic failure in the subgrade, the combined thickness of base and surface required varies directly as the logarithm of the number of load applications. Part of the data is shown in Figure 30. The slope of the line showing this relationship depends upon the type of base and surface. It is evident that the tensile strength or slab strength of the base and surface is one of the two factors which serves to protect the subgrade or basement soil.

REQUIRED THICKNESS OF BASE AND SURFACE IS A LINEAR FUNCTION OF THE STABILOMETER VALUE P_h/P_v

The relation between the quality of the subgrade and the thickness of base and surface required is one that has, and possibly always will, defy solution by mathematical analysis alone. The answer, of course, is through experimental data to correlate performance with measured quality; but this requires a solution to the problem of performing a test on the soil which will measure truthfully that quality which determines its performance.

A subgrade failure has often been described as one of "shear failure" under certain conditions of confinement, therefore, many Engineers have assumed that a simple shear test would be appropriate and adequate. However, shear resistance in a material is due to two distinct properties namely friction and cohesion and these two properties combine in very different proportions to make up the total shear resistance depending on the amount and proportion of the principal pressures involved. Furthermore, the amount and proportion of the principal pressures varies tremendously in the soil under a load. Where the confining pressures are light, cohesive resistance would be most effective; where they are heavy, frictional resistance becomes more important.

Therefore, we cannot by any simple assigned test value perfectly classify soils according to their ultimate behavior and ability to support traffic when used as a subgrade. Probably our best compromise in attempting to use a single classification would be to rate soils according to total shearing resistance at confining pressures and displacements in the neighborhood of those occurring in practice.

That portion of the resistance that is due to friction can be measured by means of the Stabilometer shown in Figure 31. It is designed for application in a laboratory where a large volume of samples, 60 or more, must be tested per day. It is not operated, as is sometimes implied, as an instrument for measuring the angle of friction and cohesion of the soil (as derived by means of Mohr's Diagram).

The resistance offered by a soil as derived from Stabilometer tests is expressed as the ratio between lateral pressure transmitted and the vertical pressure applied, at 160 psi. vertical pressure, upon a specimen 2-1/2" high and 4" in diameter at an arbitrarily set deformation. As performed this test is influenced only slightly by that portion of the total resistance that may be due to cohesion. When necessary this property is evaluated by the Cohesiometer, however for untreated soils cohesive strength can usually be neglected as a small factor of safety. Cohesive strength is important in the pavement or surface course. Figure 21. It is less important in the lower soil layers and may show much variation, depending upon density, moisture content etc.

Data has been collected from failed areas of many highways. Figure 32 represents some of the values from failed areas carrying an average amount of highway traffic. On the graph are plotted the thickness of cover material against the Stabilometer values of the underlying soils, the soils being tested at the prevailing conditions of moisture and density when the "failure" was progressing most rapidly. The heavy line then represents a boundary and conditions represented by points to the lower left are from experience likely to fail, whereas there is no record of failure under conditions represented by points to the upper right of this boundary.

This line indicates that, within the range of subgrades encountered, the thickness of base and/or surface required is a linear function of $1-P_h/P_v$ Figure 19, which is proportional, theoretically, to the maximum shearing stress divided by the major principle stress. In order to avoid confusion with misleading concepts of "bearing value" or "shear stress" the ability of the soil material to resist deformation has been designated as the "resistance value" "R" which is computed from Stabilometer data as follows:

$$R = (1 - P_h/P_v)\ 100$$

Where R = Resistance value of the material tested
P_v = the applied vertical pressure (typically 160#/sq.in.
P_h = the transmitted horizontal pressure (Stabilometer reading)

REQUIRED THICKNESS OF BASE AND SURFACE IS PROPORTIONAL TO THE FIFTH ROOT OF THE COHESION

Deflection measurements made on Asphaltic Concrete sections of several test tracks show that the resistance to deformation increases appreciably as the temperature of the surface decreases, the effect of the decreasing temperature being, in turn, to increase the rigidity or tensile strength of the bituminous binder. In addition, the performance of various types of bases indicated

RELATIONS OF LOAD REPETITIONS TO THICKNESS OF BASE AND SURFACE REQUIRED
FOR VARIOUS TYPES OF BASES

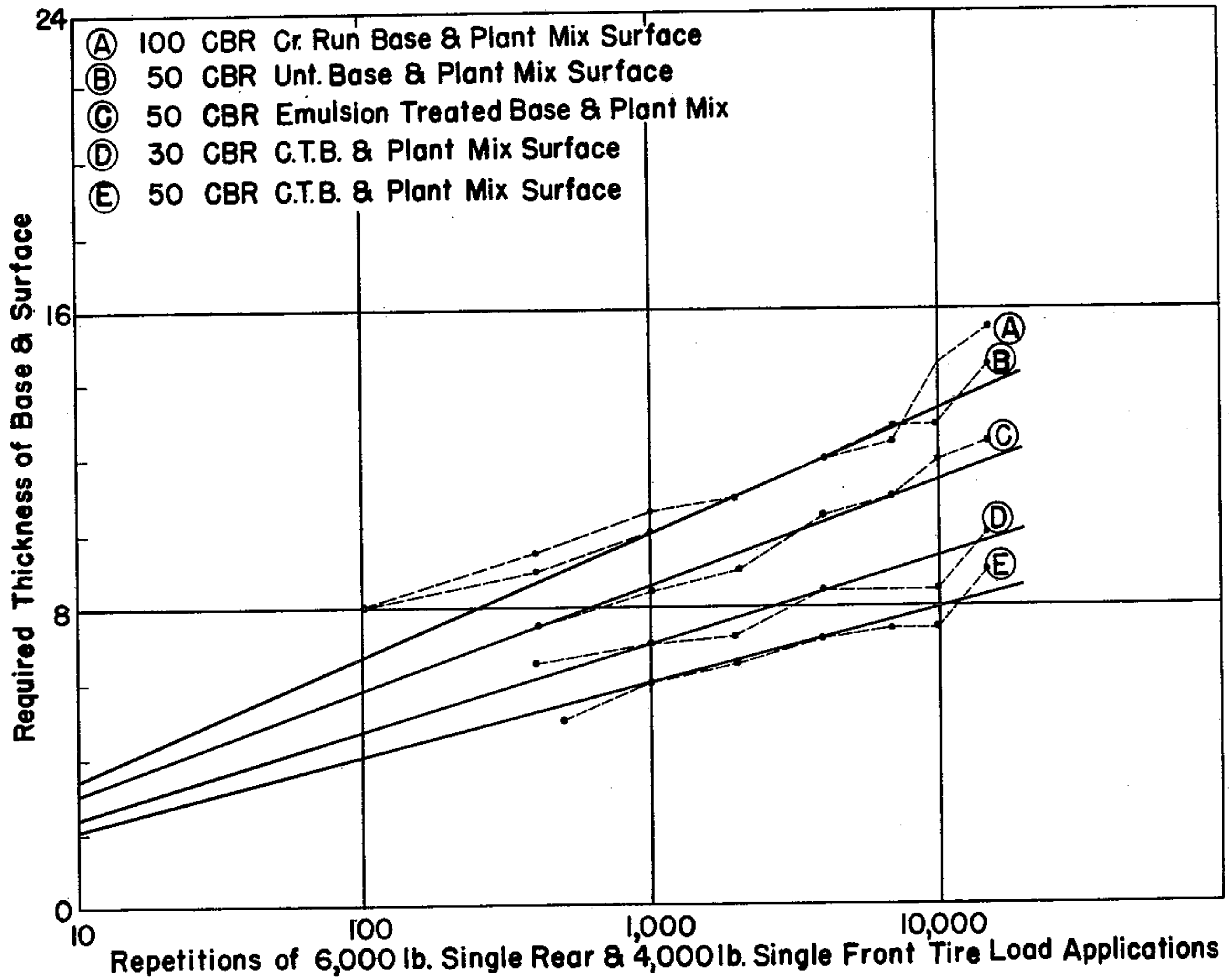
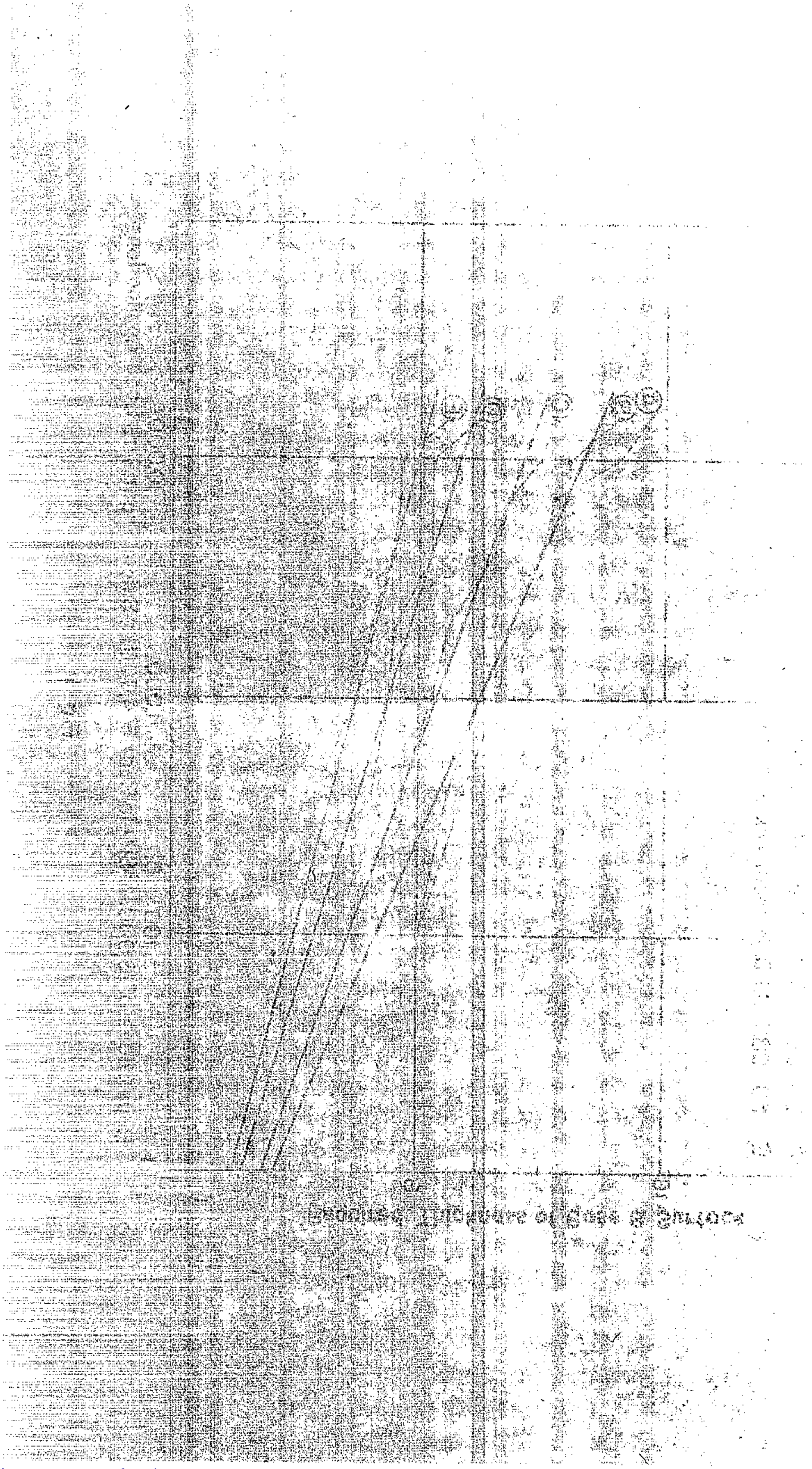
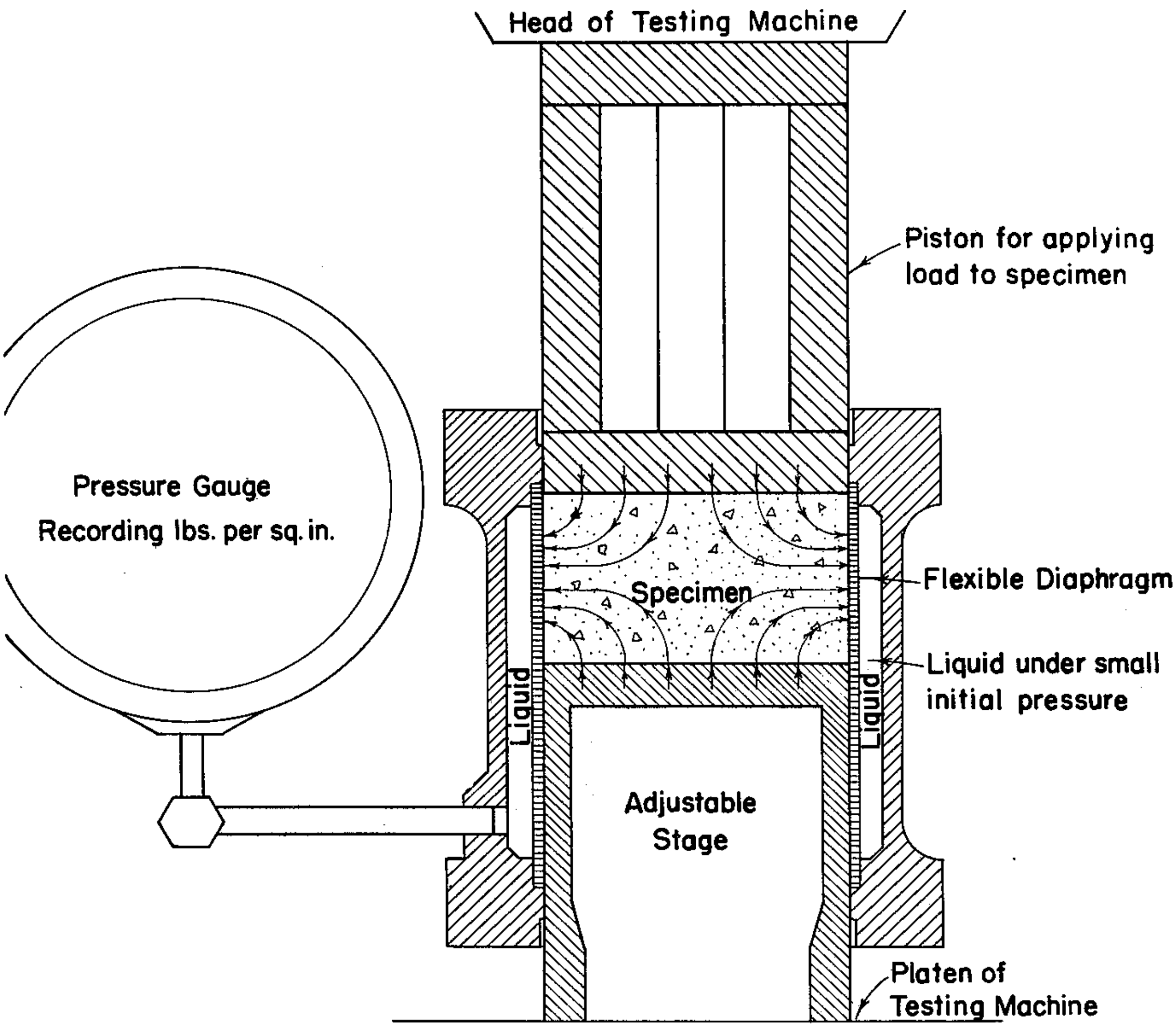


Fig. 30


Required Thickness of Base & Surface

DIAGRAMMATIC SKETCH
of the
HVEEM STABILOMETER

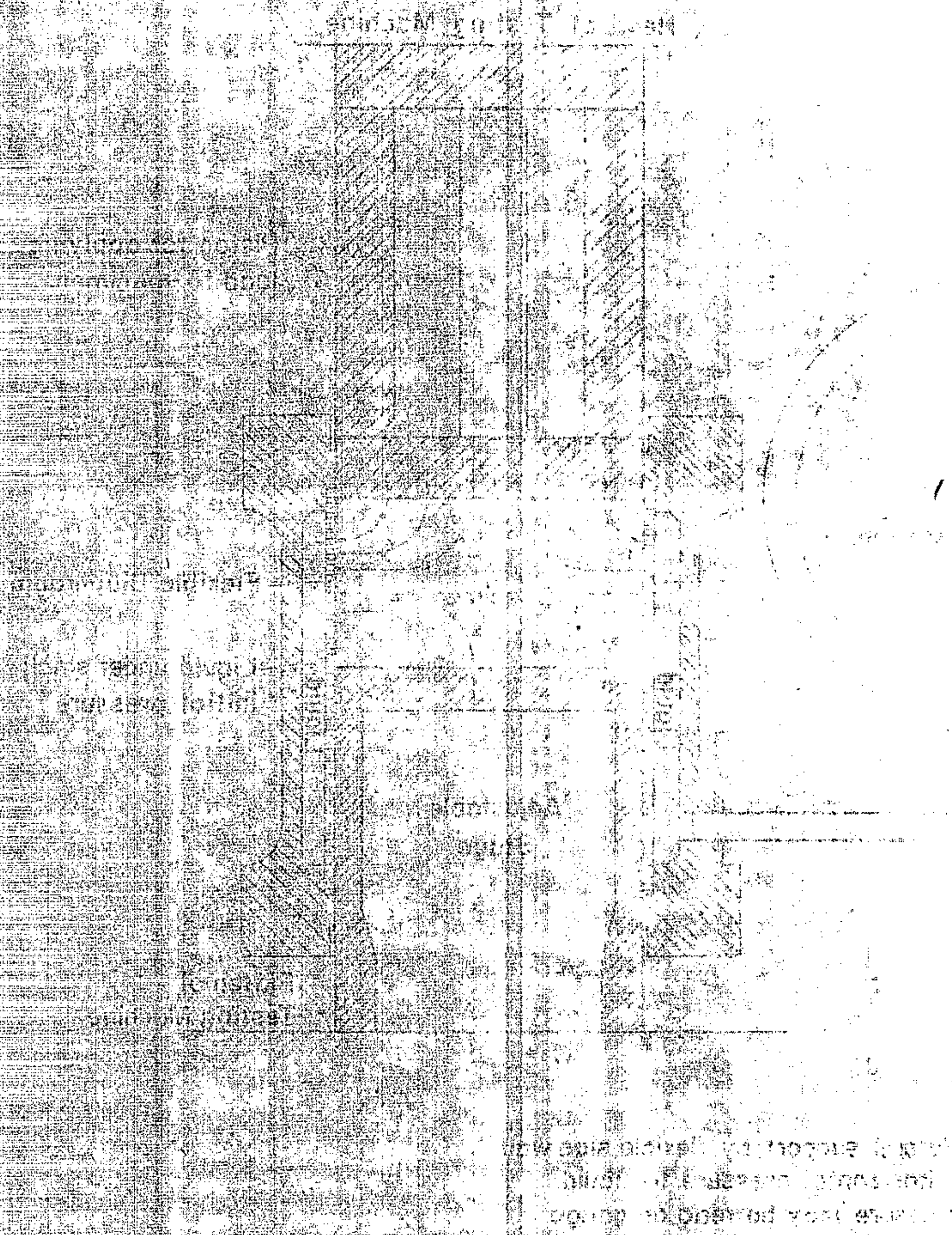


Note:

Specimen given lateral support by flexible side wall which transmits horizontal pressure to liquid. Magnitude of pressure may be read on gauge.

Fig. 31

DIAGRAMMATIC SECTION
of the
HVEEM STABILOMETER



Stability of Base or Subgrade
Vs.
Thickness of Cover for Failed Areas of Highway

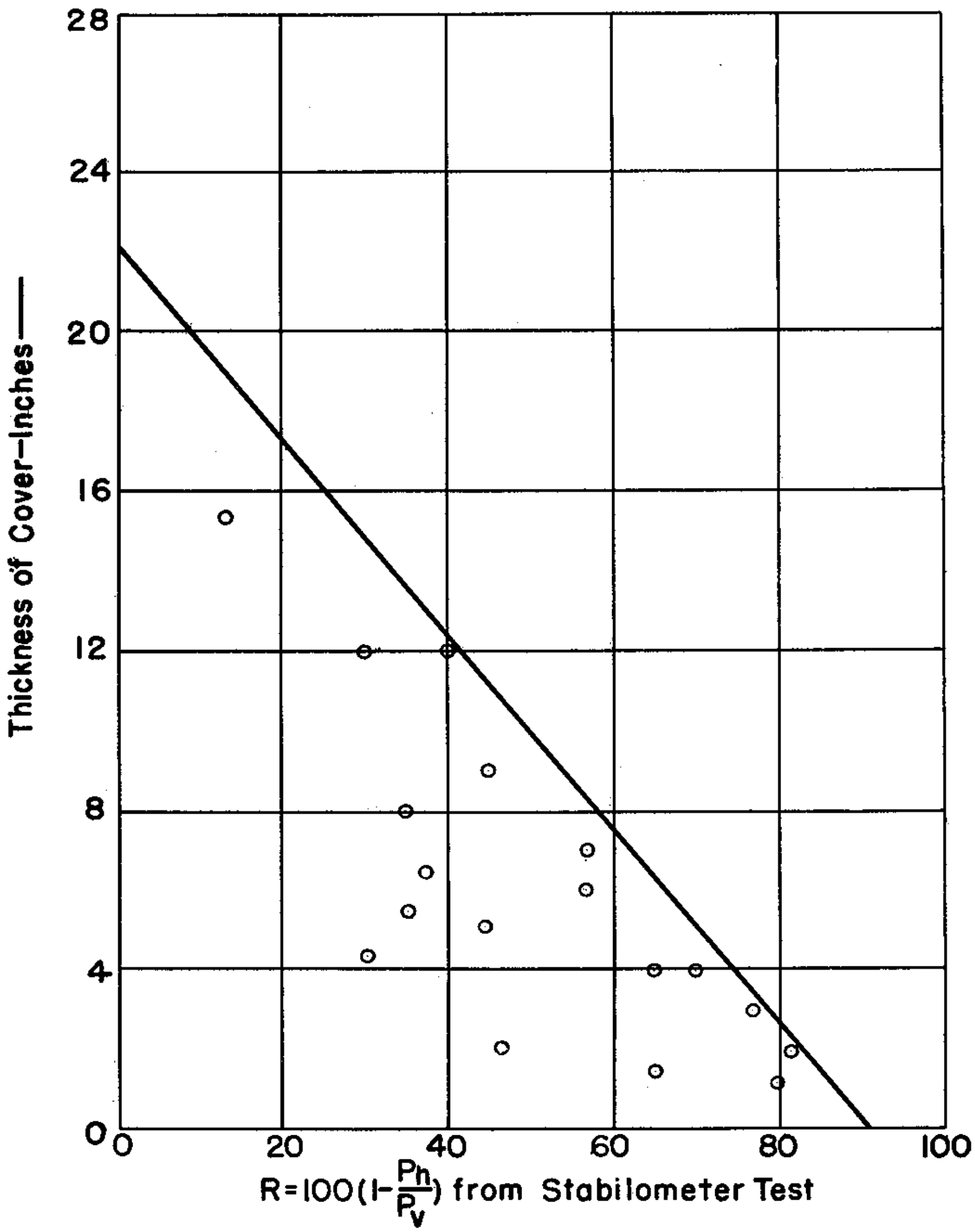


Fig. 32

that the minimum thickness that would stand up under traffic varied in inverse proportion to the fifth root of the tensile strength as measured by the Cohesiometer, Figure 37. From page 19 and from Table I and II, Figure 27, we can conclude that the effect due to the slab strength of the base and surface is most pronounced outside the area of the load, see also Figure 3.

As the deforming subgrade in this area tends to bend the base and surface upward, it is advantageous to have the upper half of the base and surface of material capable of resisting tensile deformation and the bottom half of material capable of resisting compressive deformation. Therefore, the top part of the base and surface must possess cohesive strength in order to resist tensile deformation. When the accumulated effect of forces transmitted by a plastic soil or base becomes sufficient to rupture the surface, rapid failure then takes place and it is the ultimate tensile strength of the upper layer that determines this point of gross failure for a rigid pavement. However, up to this point, the ultimate tensile strength of the surface has not been reached and while it is still intact, the resistance offered by the base and surface to the deforming effect of traffic is related to its dynamic modulus of elasticity.

In the case of a plastic base and surface, such as asphalt concrete, the magnitude of the tensile resistance will depend greatly upon the speed of the load application and will not increase appreciably as the subgrade deforms, but with a more rigid or elastic type of base and surface, such as portland cement concrete or cement treated base, the resistance to any upthrust increases rapidly until the point of fracture is reached. Although this seems to indicate a tremendous advantage in favor of the more rigid bases, it may be offset by other factors. Rigid pavements are particularly susceptible to fatigue failures or distress due to warping, pumping action, etc. that lie outside the province of structural adequacy as defined on Chart Figure I, problem #2 i.e. ability to support loads over a plastic soil. See problem #3 of Chart Figure 1 for factors involved in fatigue failures.

The value of a base and surface therefore will depend upon a combination of (1) unit weight, (2) plasticity, (3) dynamic modulus of elasticity, and (4) the ultimate tensile strength or "slab effect" of the upper half. Test track data have shown a fair correlation between minimum thickness of base and surface required and the tensile strength of the surface or base which ever is stronger.

Figure 30 shows the relation between thickness of base and surface at the point of failure for five different types of base and surfacing. For all types the thickness of base and surface was found to be proportional to the logarithm of the number of load repetitions and the slope of the curves for the different types of base, inversely proportional to the 5th root of the Cohesiometer value, which in this case is a value directly proportional to the modulus of rupture of the base material or the surface type when measured under certain specific conditions of test (The Cohesiometer for example).

Summary

From the foregoing, we can state, based upon all evidence available, that the required thickness of cover material necessary to protect the underlying soil from plastic failure is proportional to (1) average tire pressure (2) the square root of the effective imprint area, (3) the logarithm of the load repetitions, (4) the function (P_h/P_v-.1) (derived from Stabilometer tests) and (5) inversely proportional to the 5th root of the tensile strength of the pavement, base and surface (as derived from Cohesiometer values or modulus of rupture determinations).

The formula then becomes:

$$T = \frac{(K\ P\ \sqrt{a}\ \log r)\quad (P_h/P_v - 0.10)}{\sqrt[5]{c}}$$

where T = Thickness of cover (Base and Pavement) in inches
K = .0175 for best correlation but without any factor of safety. For design purposes it is suggested that K - .02
P_h = transmitted horizontal pressure in the Stabilometer test (#/sq.in.)
Pv = applied vertical pressure in the Stabilometer test (typically 160#/sq.in.)
P = effective tire pressure (#/sq.in.)
a = effective tire area (sq.in.)
r = number of load repetitions
c = tensile strength of the cover material as measured by the Cohesiometer in gms. per sq. in. (approximately = Modulus of Rupture X 45.4)

Figure 33 shows an alignment chart prepared in accord with the above formula, however, it is plotted using a value of K = .02 in order to include a factor of safety. From it may be estimated the required thickness of base and surface necessary to prevent failure in the subgrade (likewise thickness of pavement or surface to prevent failure of the base). Figure 34 illustrates the potential movement in one or more layers. Referring to chart, Figure 33, a line passed through selected points on any two of

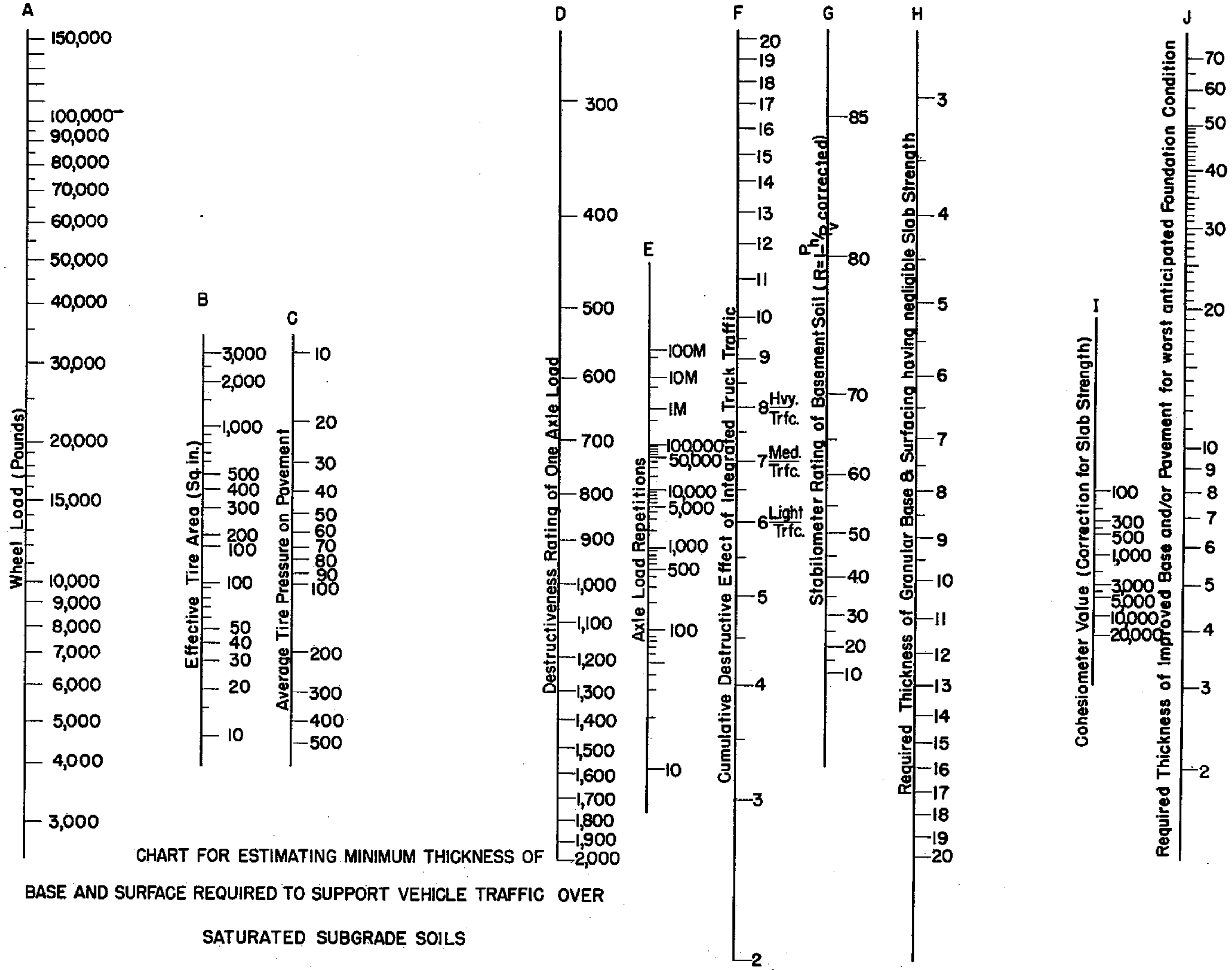


CHART FOR ESTIMATING MINIMUM THICKNESS OF BASE AND SURFACE REQUIRED TO SUPPORT VEHICLE TRAFFIC OVER SATURATED SUBGRADE SOILS

FIG. 33

[illegible]	[illegible]	500	50
2000	400	[illegible]	[illegible]
8000	200	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]

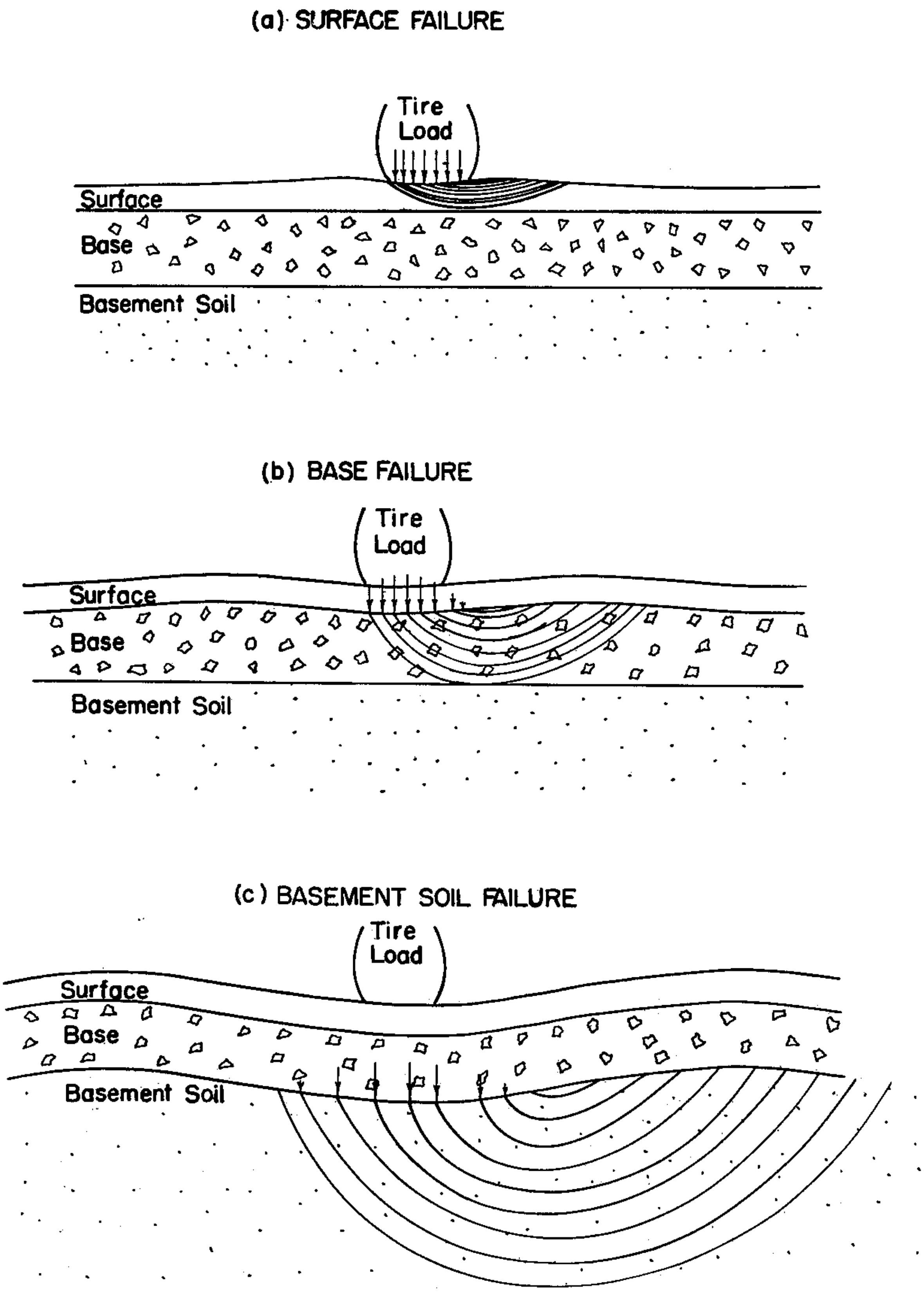
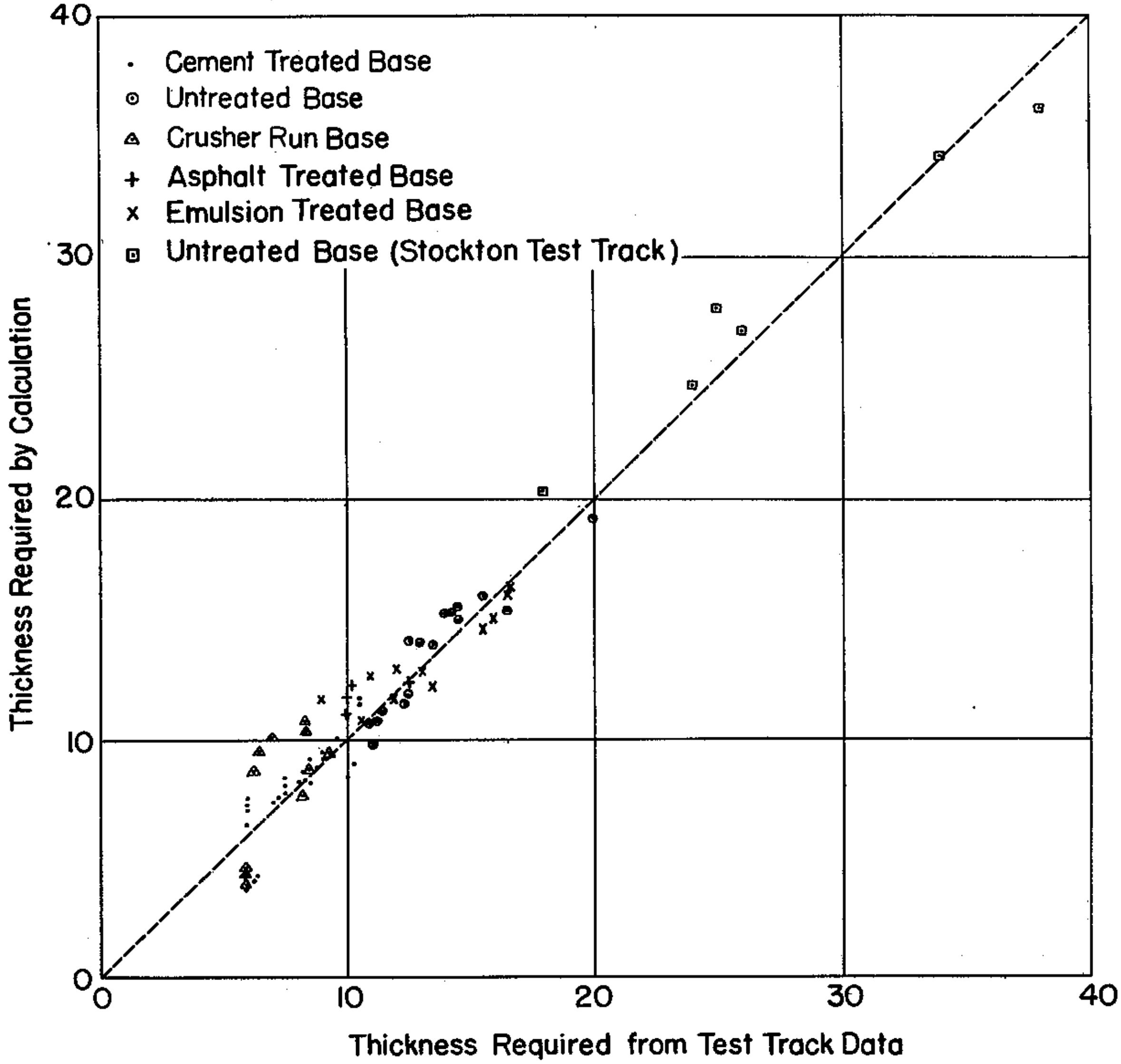

(a) SURFACE FAILURE
Tire
Load
Surface
Base
Basement Soil
(b) BASE FAILURE
Tire
Load
Surface
Base
Basement Soil
(c) BASEMENT SOIL FAILURE
Tire
Load
Surface
Base
Basement Soil

Fig. 34



Cement Treated Base
Untreated Base
Crusher Run Base
Asphalt Treated Base
Emulsion Treated Base
Untreated Base (Stockton Test Track)
Thickness Required by Calculation
Thickness Required from Test Track Data
0
10
20
30
40

Fig. 35

the scales A,B and C will intersect Scale D at a point showing the destructiveness rating of that particular combination of wheel load and tire pressure. A line passed through this point on Scale D and the appropriate point on Scale E (which shows the number of repetitions of this load) will intersect Scale F at a point showing the cumulative destructive effect of this traffic. The data upon which these scales are based are taken from test tracks in which the loads were repeatedly applied along the same lines. Actual highway traffic, even in a single lane, would be staggered somewhat and the concentration would be less severe.

Figure 35 shows the thickness calculated from the above formula plotted against the actual required thickness as found from the Brighton Test Tracks and the Stockton Test Track. The data includes single wheel loads from 6,000 to 40,000 pounds, load repetitions from 6,000 to 77,000, six different subgrades eight different bases and two types of surface.

III

PREPARATION OF TEST SPECIMENS

In the foregoing discussion it was mentioned once or twice that the resistance of granular masses to deformation under load is primarily dependent upon inter-particle friction. In fact, Hardy has stated that "friction is a characteristic of particulate matter." Therefore, it appears that the condition which prevails at the points of contact between soil or rock particles is a matter of great importance. It becomes appropriate to consider the manner in which soil or particles of mineral aggregates are brought together and especially the force with which these particles are pressed into intimate contact. This force or contact pressure will have an important influence on the resistance to further movement. Therefore, if a significant laboratory test is to be developed, it becomes mandatory that test specimens must in all essential respects reproduce the structural conditions of the prototype.

Thus, we must be concerned over the methods used in compacting a test specimen of soil, crushed stone base or bituminous surface mixture. Even a casual consideration of field compaction equipment and current specifications will furnish evidence that pressures exceeding 300 lbs. per sq. in. are not common and furthermore, it is well known that heavy construction equipment equipped with pneumatic tires will produce a very high compaction i.e. density of embankment soils and granular base materials. This effective compaction is accomplished with tire pressures often less than 100 lbs. per sq. in. It seems obvious, therefore, that laboratory compaction methods using straight compression loads equivalent to two or three thousand pounds per square inch represent a tremendous increase over any load that has thus far been employed on actual construction. Samples compacted under these excessively high pressures cannot be expected to bear much structural resemblance to the same materials in place on the road. Usually, the common excuse given for using such high compaction pressure is that "it is necessary in order to develop proper densities," that is, to produce laboratory specimens having weights per cubic foot similar to materials in place in the roadbed.

It can easily be shown that the "density" of a granular mass is one of the least reliable and least informative of all determinations which can be made. This expression is simply the ratio between the absolute volume of solid material and the absolute volume of voids for a given over-all volume of material. Tests may be performed on soil specimens which have been compacted by several different methods, for example, by tamping, pounding, vibration, by straight compression loads, and combinations of these various methods.

Any or all of these compacting procedures can be utilized to produce some given density and based upon density comparison alone the results might be judged to be similar. However, with many soils or aggregates, the internal structure or the particle arrangement may vary considerably without any significant change in density. In the majority of cases the resistance to further displacement is increased as the compaction load is increased even though the relative density may not show a proportionate increase. Therefore, before any intelligent selection or discrimination can be made between methods of testing and measuring important properties of the soil, it is first necessary to make sure that the specimen tested is a reasonable model of the prototype. Laboratory test results are liable to be very deceiving when performed on a test specimen wherein the resistance has been built up by excessive compaction far beyond the state now attainable with construction equipment.

Sand or rock particles having considerable surface friction are not easily forced or pressed into a close fitting state. The action of a rolling wheel tends to displace granular materials in a lateral direction and compaction is accompanied in large measure by a shifting of particles, usually in a more or less horizontal direction. This movement has a tendency to produce laminations which may be observed in soil specimens and it has also been observed that particles of coarse stone in asphalt paving mixtures tend to come to rest with their long axes in a horizontal position. All of this means that while relatively high densities are often achieved under field conditions, it does not necessarily prove that high pressures have been exerted between adjacent particles of the aggregate or soil. Any attempt to duplicate field conditions, by compressing granular soil materials in a steel mold using a full area load operates at a great mechanical disadvantage and requires that very great pressures must be employed in order to overcome the internal friction and thus to reproduce the field density. This is due to the fact that when under pressure from all sides the particles are unable to shift or slide into the closest fitting pattern. It may therefore be stated that density as an end product of careful gradation and the close fitting of particles may have a quite different significance when compared with an equal density which has been developed as a result of heavy compaction pressures which force particles into place.

If the resistance to displacement is influenced by friction and the magnitude of frictional resistance depends upon pressure, it then becomes clear that a granular mass forced into a certain high degree of density by means of pressure alone will offer a much greater resistance to displacement than will the same mass brought to the same density by other means which involve less pressure. The truth of this premise can be demonstrated by subjecting samples of gravel and clay mixtures to various compaction techniques. It will be found that many materials will have a greater resistance to displacement when the specimens have been compacted by heavy static pressure than will be the case when compacted by a series of light loads applied over small areas of the specimen surface. This latter action will permit the particles to move about until maximum density is achieved without abnormal inter-particle pressure.

It is, of course, not a simple matter to manufacture a soil test specimen in the form of a cylinder a few inches in diameter which will reflect the same particle relationship in the structure as is developed in the field by construction equipment. Nevertheless, a close approximation can be obtained by a species of kneading action which applies a moderate load to only a small sector of the surface at a time while the balance of the surface is unrestrained. If impact is eliminated it is possible to duplicate field densities through the use of pressures measuring from three to five hundred pounds per square inch (which are not far beyond those normally employed in construction). The validity of this assumption has been independently proven for bituminous paving mixtures and for untreated soils and gravel mixtures. Early attempts to correlate stabilometer test results on core specimens cut from asphaltic pavements with identical mixtures compacted in the laboratory indicated that considerable discrepancy existed. A better correlation was secured when the laboratory specimens were compacted in the manner indicated above even though no consistent change in density was noted.

The tabulation in Figure 36 illustrates some of the differences in resistance values which were found to exist in samples of crusher run base taken from a section of road which was giving evidence of distortion under moderate traffic. Attention is directed to the marked differences between samples A-2 and A-3 (so far as resistance value is concerned) without any measurable change in either moisture content or density. The value "R" equals 17 is consistent with the unstable condition observed. In contrast specimen B was taken from a stable section. While the low moisture content undoubtedly was partially responsible for the good condition, this condition is reflected by the "R" value of 86.

Samples C-1 to C-6 illustrates the effects on the resistance value "R" of variations in the amount of fine material and in the corresponding moisture content equivalent to saturation.

TESTS ON MATERIAL FROM FAILED AREA

Test No.	Description	Stabilometer Value (R)	Moisture Content %	Dry Density #/cu.ft.	Pass #200
A1	Condition in place	-	6.7	145	12
A2	Compacted by method described. Moisture and grading as received	17	5.9	146	12
A3	Compacted by 2000#/sq.in. static load. Moisture and grading as received	75	5.9	146	12

TESTS ON MATERIAL FROM UNFAILED AREA

Test No.	Description	Stabilometer Value (R)	Moisture Content %	Dry Density #/cu.ft.	Pass #200
B1	Condition in place	-	3.7	149	6
B2	Compacted by method described. Moisture as in place on road	86	3.7	145	6

TESTS SHOWING VARIATION OF STABILOMETER VALUES WITH PERCENTAGE PASSING 3/8" SIEVE

Test No.	% Passing 3/8"	Stabilometer Value (R)	Moisture Content %	Dry Density #/cu.ft.	Pass #200
C1	70	24	9.3	142	15
C2	60	24	7.9	143	13
C3	55	40	5.8	147	12
C4	50	50	6.0	147	11
C5	45	75	5.3	148	10
C6	40	88	4.8	148.5	9

Fig. 36

The initial state of compaction and density of the soil is the result of energy applied from outside the material; namely, by compaction equipment. The ultimate density after the passage of time may be influenced by internal forces such as the expansive action of certain soil material in the presence of water, therefore, in the preparation of a truly representative soil specimen we must first compact the material in a manner which will reproduce the internal structure typical of the roadbed immediately after construction. For most highway and airport pavements it is then appropriate to assume that water will have access to the basement soil and it is necessary to anticipate the tendency of the soil to swell or expand to the extent that may be expected in the field. The forces which tend to expand the soil during the absorption of water are not unlimited and it follows that any expansion will be inhibited or counteracted by opposing forces such as the weight of superimposed pavement and base layers. If we can measure the amount of expansive force which the soil will generate while soaking up water, it is a simple matter to calculate the load which is equivalent to the expansion pressure, which in turn indicates the unit weight and therefore, thickness of cover material required to prevent expansion beyond a certain point.

The question of volume change or magnitude of linear expansion of a soil is not in itself a serious matter and could generally be ignored if it were not for the fact that expanded soil has a larger void space and therefore, can take up more water and as a consequence will be less stable and have a lower capacity to sustain loads. A description of the essential equipment and the test procedures is set forth in Appendix I submitted herewith. A typical compaction machine is illustrated together with an apparatus for measuring the expansion pressure.

The foregoing comments on compaction procedure and the reference to the swelling phenomena in soils are in accord with the fundamental idea that the materials being tested in the laboratory must be representative of the worst condition which will be typical of the materials in place for a number of years after construction. The Engineer should decide whether "worst condition" means saturated or at a lesser moisture content. This means that the moisture content and condition of the test specimen can and should be adjusted to compensate for local conditions, rainfall, frost action, drainage, etc.

Having subjected the soil sample to a compaction process that will develop the same sort of inter-structure as is typical of soils compacted under construction equipment and having permitted the sample to have access to moisture and the resultant expansion pressure measured, the next step is the determination of the "stability" or resistance value of the soil. The term stability means the capacity to resist displacement and is the property which most Engineers have in mind when speaking of supporting power or bearing value. The Stabilometer offers a means for subjecting a sample of compacted soil or granular material to a controlled test load with means for measuring the lateral pressure generated. The Stabilometer, Figure 31, is probably familiar to many Engineers. Variations of this instrument have been designated as devices for measuring triaxial shear. The modifications of the apparatus which have been used for triaxial testing, however, are usually employed in a manner somewhat different from the Stabilometer test procedure.

Compacted specimens are placed in the Stabilometer and subjected to a vertical load reasonably typical of the service conditions that are anticipated. The capacity of the soil mass to transmit pressure is measured in the Stabilometer and it is then possible to compare the ratio between the vertical applied pressure and the horizontal transmitted pressure as indicated by the expression $R = (1 - P_h/P_v) \times 100$. See Appendix I.

IV

DESIGN PROCEDURE

An attempt was made in Chapter I to analyze the pavement problem and to describe the component parts of an adequate pavement design. In part I B consideration was given to the intimate circumstances surrounding the direction and pattern of particle movement in a soil mass under load. Chapter II discussed certain mathematical relationships between tire pressures, load areas and frequency of load repetition as related to soil and pavement properties. Chapter III is confined to a discussion of the problem surrounding the preparation and testing of representative soil specimens. It is now necessary to indicate the manner in which the formulas and test data can be utilized to provide an economical and adequate structural design so far as the ability to support loads is concerned.

The formula at the end of Chapter II represents the closest agreement between the theoretical factors and the observed performance and necessarily includes the expression ($P\sqrt{a}\log r$) to indicate the ultimate destructive effect of traffic. However, it will require further study and development to convert this expression into a useful form which could be applied directly to a known traffic distribution on an existing highway route.

In order to develop a design procedure that could be applied to current data, the formula has been modified to accomodate traffic values computed in terms of equivalent wheel loads. A procedure for this computation was adopted by the California Division of Highways and reported in California Highways and Public Works, March, 1942. This article listed a group of 6 constants indicating the relative destructive effect of wheel load groups ranging from 4,500 pounds to 9,500 pounds. A modification of this method has been developed by Mr. A. M. Nash, Design Engineer, for more convenient application to traffic census data. The procedure consists of summarizing the accumulative destructive effect of the anticipated traffic by means of constants applied to the current traffic count and it is necessary that the traffic census be secured in sufficient detail to indicate the typical distribution pattern of the various truck types. Having found that the traffic on a certain road follows a fairly constant pattern so far as the number of each type of vehicle is concerned, it has also been established that in the over-all pattern each typical vehicle of a certain type could be expected to carry a certain wheel load. From these data, constants were developed for each type of commercial vehicle dependent upon the number of axles and having information on the relative percentage of each type on a given road, it is then a simple calculation to summarize the total into the equivalent wheel load repetitions which are based upon a standard load of 5,000 pounds.

The following tabulation will give an example of the constants used and the results of the calculation applied to a typical traffic census of the commercial vehicles.

SAMPLE CALCULATION

No. Axles	EWL Constants	Current Average Daily comm. vehicles	Product of Columns 2 & 3
2	300	774	232,200
3	700	212	148,400
4	1,400	68	95,200
5	2,100	118	247,800
6	1,600	112	179,200
	Total Annual Design EWL Repetitions		902,800

Assuming an anticipated increase in commercial traffic of say 50% during the ten years following construction, the total wheel load repetitions at the end of ten years will equal $10 \times 902{,}800 \times \frac{1+1.5}{2}$ or 11.3 million. The EWL constants contain all of the factors involved and the foregoing represents the value to be used in design without further modification.

The final value derived by the above calculation may then be used in the thickness design formula by substituting .12 log EWL for $(.02\, p\, \sqrt{a} \log r)$.

Having thus secured a usable traffic factor for the particular highway to be designed, it is then only necessary to know the resistance value or rating of the underlying basement soil and the cohesiometer rating for bituminous pavements or the modulus of rupture value of the base and surface combination when rigid types are concerned.

The principles involved in preparing a satisfactory specimen were discussed in Chapter III and the details of a test procedure is described in Appendix I, together with a typical example.

Briefly, test specimens are prepared under a type of compaction apparatus and a magnitude of load typical of common construction practice. Trial test specimens are prepared at different degrees of density, each saturated with water, and obviously, the amount of water required to produce saturation will vary inversely according to the amount of compaction or density achieved.

After test specimens have been prepared representing three different states of density and moisture content, they are placed in an apparatus for measuring expansion pressure and the load required to restrain expansion is noted in each case. In virtually all cases, the expansive force is greater with the denser specimens of relatively low moisture content. The less dense specimens, having the voids filled with water, have a reduced capacity for further expansion. However, as the initial compaction is reduced and the amount of water included in the specimen is increased, the ability to sustain loads is usually diminished.

After the samples have registered the potential expansive force, Stabilometer tests are performed in order to determine the relative resistance value under the three different states of moisture and compaction. As the amount of protecting cover (which may include subbase, base and pavement) will not be known until after the resistance value has been calculated, the final solution will require the plotting of two curves as the most simple means to determine the thickness and strength of cover material that will satisfy the conditions for both problem one and problem two as stated in chart, Figure 1.

After the resistance value for the layer (basement soil, subbase or base course) has been determined from the Stabilometer tests, the necessary cover thickness to prevent plastic flow is estimated by means of chart Figure IV of Appendix I. These values are noted for the appropriate EWL values representing the weight of traffic and for the type of pavement or base construction contemplated. These values are used to complete the curve as shown in Figure V of the Appendix in order to determine the point at which the thickness of pavement required to support traffic is also sufficient in weight to restrain any further expansion of the soil and thus will insure that the moisture content of the underlying layer cannot exceed the condition indicated.

A few comments on the significance of the values shown in Appendix I, Figure IV may be in order. Several more or less elaborate versions of this chart have been prepared (for example see Figure 33). It is desired to point out that the Cohesiometer values shown on Scale D for the several types of pavement and base construction illustrated are only broad approximations of the material in place on the road. A certain amount of judgment must be exercised, however, in utilizing these apparently simple numerical values, as for example, there can be no completely accurate comparison established between the tensile strength or Cohesiometer value of such distinctly different materials as asphaltic concrete and portland cement concrete. The effective tensile strength of a bituminous pavement will vary markedly with time, temperature, speed of traffic, grade of asphalt and density of the pavement.

While there is an arbitrary relationship indicated on the chart Scale D between Cohesiometer values and the modulus of ruptures this parallel exists only where rigid materials such as concrete or cement treated bases are concerned. The modulus of rupture concept is not applicable to ductile materials such as asphalt paving test specimens.

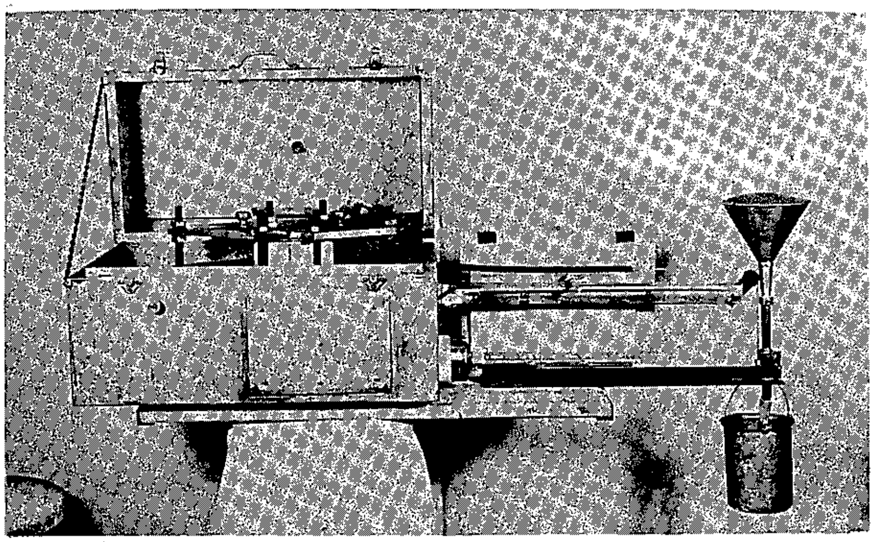
For standing loads, bituminous mixtures have but little more capacity to support loads than the same thickness of untreated crushed stone. For rapidly moving traffic, however, this resistance may show a really tremendous increase. Therefore, the relative strength values shown on Scale D, Figure IV can only be assumed to be valid for normal highway speeds.

In view of the variations which exist with the range covered by each type of construction material, it is not necessary or practicable to test each particular design for use, instead average values have been assumed, based upon tests performed on specimens taken from the roadway and these values are not properly indicated by tests made upon laboratory specimens which have not been subjected to the same conditioning of time and traffic.

It is believed that the foregoing presentation has recognized or at least mentioned most of the important factors which bear directly or have an influence on the problem of pavement design. As a satisfactory pavement must have a number of properties or characteristics, it is impossible to cover all by a single test procedure or by a single criterion of design. It is hoped that the testing procedures and the design formulas herein set forth will make it possible to apply a reasonably uniform and common technique in the attack on the problem of structural design in order to provide a sufficient thickness and strength of pavement which will sustain traffic loads without excessive deformation of the underlying layers by plastic flow.

The question of fatigue action caused by the flexing of more or less rigid slabs over resilient foundations or over foundations which do not offer uniform support because of loss of materials by pumping action, non-uniform settlement, et cetera, are not considered a part of this problem. Adequate protection against failures of this latter type must be achieved from a solution to the problem designated as number three in the initial chart. A satisfactory answer to this problem must be sought through other avenues and through the utilization of other methods of testing and design.

It is desired to acknowledge the contribution of the many individuals who have either participated directly in the development work or have assisted by helpful advice and criticism.



HVEEM COHESIOMETER

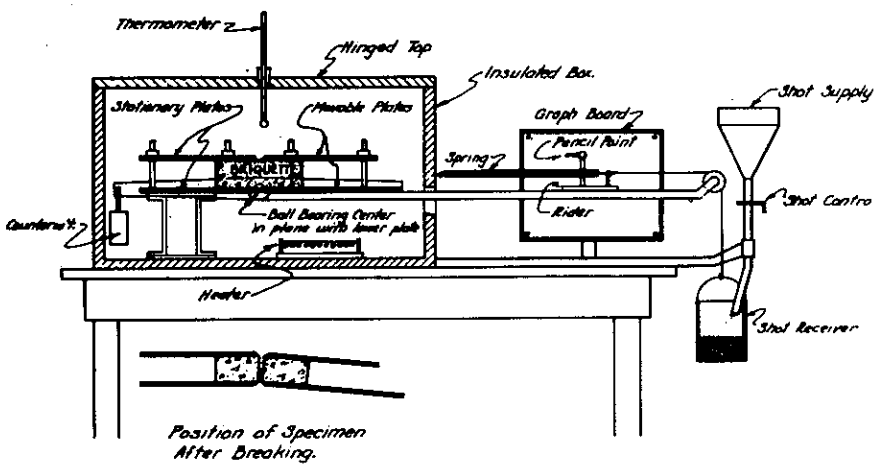


FIG. 37

The work has been carried out in the laboratory of the California Division of Highways under G. T. McCoy, State Highway Engineer, and T. E. Stanton, Materials and Research Engineer. The necessary test data were obtained from two test tracks one constructed by the Division of Highways and one by the Corps of Engineers in Stockton, California.

It is desired to acknowledge the assistance of Ernest Zube who was in charge of the testing and recording of observations of the test track; to George Sherman who assisted in the compilation of data; to Mr. D. J. Steele and Walter Liddle of the Public Roads Administration who gave much valuable assistance and advice, to Dr. K. P. Krynine who read the manuscript and offered many suggestions and constructive criticism and to many others too numerous to mention by name.

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APPENDIX I

DESCRIPTION OF THE METHOD USED IN TESTING SOILS BY MEANS OF THE HVEEM STABILOMETER AND EXPANSION PRESSURE APPARATUS

Apparatus

1. Hveem Stabilometer, Figure I
2. One special compacting apparatus to produce "Kneading" action, Figure II
3. Compression Testing Machine, minimum capacity 20,000 pounds
4. Three molds 4" inside diameter, 5" long
5. Three units, expansion pressure testing apparatus, Figure III
6. One hot plate and pan filled with paraffin
7. One set of scales suitable for weighing specimens in air and water

Procedure

All testing of soils for both plastic flow and expansion pressure is carried out when the compacted soil test specimen is in a saturated condition. Soil material used is restricted to the portion passing the 3/4" sieve.*

A sufficient amount of the minus 3/4" material to form a compacted specimen 2-1/2" high by 4" in diameter is mixed with slightly more water than is expected to produce saturation after compaction. The soil is then compacted in the special compacting apparatus shown in Figure II. This compactor consolidates the material without depending upon straight compression or damaging impact, but rather by a series of individual impressions made with a roving ram having a face shaped as a sector of a 4" diameter circle. This small size ram or tamping foot develops a definite kneading action, simulating that given to the road by rollers or by rubber-tired traffic. Compaction is achieved by 100 applications of the

*It is assumed that upon this part depends the resistance to plastic flow and the expansion pressure exerted, that all stones larger than 3/4" are floating in a matrix of the passing 3/4" and are not sufficient in quantity to greatly affect the plasticity or expansion pressure of the entire mass.

tamping foot applied each time to a different sector of approximately 3-1/2 square inches. At each application, pressure increases gradually to a maximum of 350 pounds per square inch. After removing from the compactor, the specimen is subjected to a pressure of 500 pounds per square inch over the whole of the area. (At this point water should be exuded from the soil as evidence that enough moisture is present to produce saturation). Two more specimens containing greater amounts of water (with correspondingly lower density) are similarly molded but with less compactive effort and, after allowing sufficient time for any rebound, the three are placed in an expansion pressure testing apparatus where the variation in expansive force exerted by swelling under water is determined for the three different initial conditions of moisture and density.

The expansion pressure testing apparatus, as shown in Figure III, is a device in which the compacted soil specimen is confined under a perforated disc covered with water and the pressure exerted by the soil at the point of incipient expansion is measured by means of a mechanism similar to a proving ring. A total force of 6-1/4 pounds or a pressure of .5 pound per square inch requires only .001" movement in order to register on the gauge; therefore, the pressure is measured at virtually constant volume.

After 24 hours in the expansion pressure apparatus, the final pressures are recorded, the specimens removed and tested in the stabilometer.

The stabilometer is shown in Figure I. It is an instrument for subjecting a 2-1/2" x 4" diameter specimen to triaxial compression. Base and subgrade soils are evaluated according to the expression

$$R = (1 - P_h/P_v)\ 100$$

Where R = Resistance value of the material tested
P_v = the applied vertical pressure (typically 160#/sq.in.)
P_h = the transmitted horizontal pressure (stabilometer reading)

Theoretically this expression is directly proportional to the maximum shearing stress divided by the major principal stress. It has been found by correlation of field and experimental data that, other things being equal, the thickness of cover* required over the

*Cover material includes all layers above the soil in question. For example, "cover" may include subbase, base and surface courses when the basement soil is being considered. "Cover" would include only base and surface when the subbase material is being tested. Similarly when the base is being evaluated, "cover" would mean the bituminous surface or pavement alone.

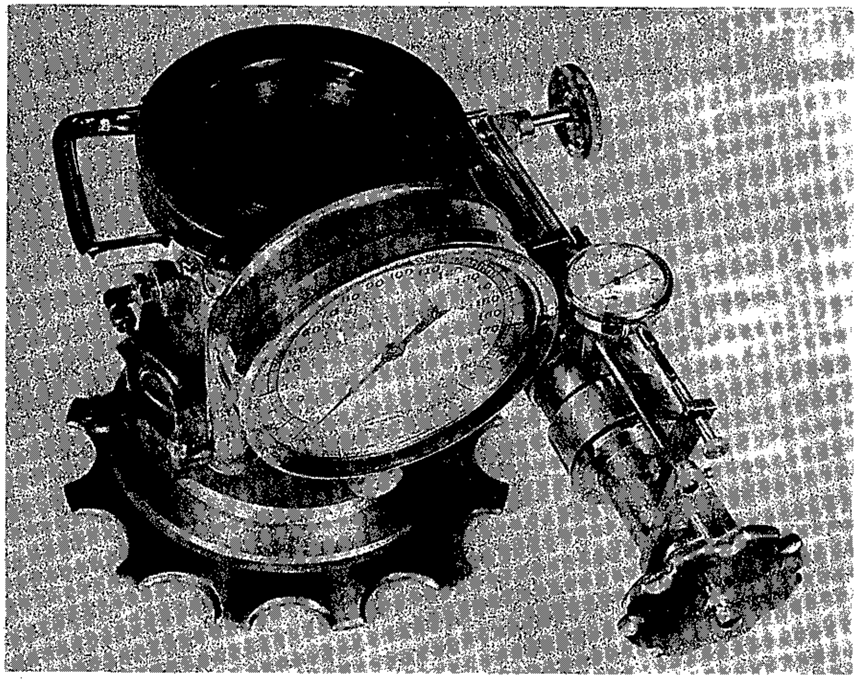


Figure I

Hveem Stabilometer which measures the transmitted horizontal pressure resulting from an applied vertical pressure.

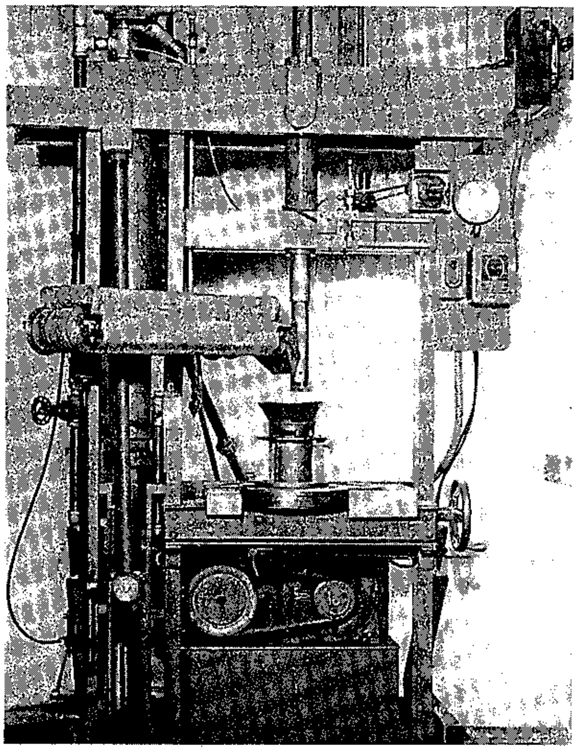


Figure II

Special compacting apparatus which compacts with a slight kneading action simulating that of rollers and rubber-tired traffic.

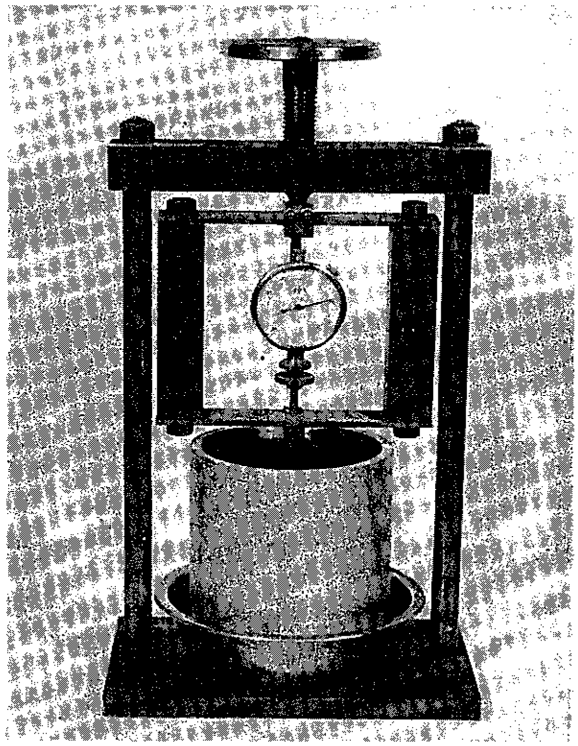


Figure III

Expansion Pressure apparatus which measures the pressure generated by swelling soils when held at practically constant volume in contact with water.

soil layers tested is a linear function of this expression when the value is measured at the equilibrium condition of moisture and density for the soil in place.

After the stabilometer test is completed, the briquette is broken in half and one part used for density determinations (by coating with paraffin and weighing in air and in water) and the other half is used for a moisture determination.

Having determined the "R" values and expansion pressures of the underlying soil for the several states of moisture and density and knowing the estimated traffic and type of surfacing to be used, the design thickness of cover is evaluated as follows:

First, by means of Figure IV the thicknesses of cover necessary to prevent plastic flow of the underlying soil for the several states of saturated moisture conditions are determined. Second, the thicknesses or weights of cover material necessary to restrain this soil (by preventing expansion and thus maintaining it in the several states of moisture and density) are calculated from the expansion pressures. From the two relationships, a balance point may be selected in which the thickness of cover material is sufficient to prevent plastic flow under a given weight of traffic for a certain moisture density condition, and this cover is also of sufficient weight to counterbalance the expansion pressure exerted by the foundation soil (or any other layer) for the same moisture and density condition.

The procedure may be illustrated by the following example:

It is desired to determine the necessary thickness of crusher run base and AC surfacing to be placed over a given subbase* in order to support fairly heavy commercial traffic (which has been calculated as being equal to Traffic Index - 8 on Scale B, Figure IV)**

*The same procedure would be followed in estimating the total thickness required over the underlying basement soil, embankment or in cut sections.

**Traffic Index of 8 on Scale B represents 3,500,000 equivalent 5000# wheel load for the life of the road calculated from a traffic count according to the method described in California Highways and Public Works, November 1941.

The results of tests on the saturated specimens of subbase material are shown in the following table:

	a	b	c	d	e	f
	Specimen Condition When Tested			Necessary Cover Thickness		Necessary Cover Thickness
Sub-base Spec	Moist. %	Dry Density #/Cu.Ft.	Resistance Value (R)	(To Support Traffic)	Exp. Pres. #/sq.in	(To Restrain Expansion)
A	12.2	120.5	23	14.5	.2	2.7
B	11.5	122.5	35	12.0	.4	5.4
C	10.8	124.0	57	7.5	.8	10.7

Following the resistance values, Column c, which were determined by means of the stabilometer, Column d shows the thickness of base and AC surface (Cohesiometer Value = 500) required for heavy traffic, when the subbase is in the condition indicated in Columns a and b. Pavement and base thickness values are determined by means of Figure IV.

Values are shown in Column f for the thickness of base and surface (based on 130#per cu.ft.) that would be necessary to counteract the expansion pressure and, thus, maintain the subbase at the corresponding density and moisture content even in the presence of free water.

Plotting these two sets of thickness values against moisture content, Figure V, Curve (d) shows the thickness of base and surface necessary to protect the subbase against failure under traffic by plastic flow and another curve (f) shows the thickness necessary to counteract the expansion pressure and, thus, prevent the subbase from taking up more water. The ordinate at the intersection point of these curves then shows the most economical design thickness that will keep the subbase from expanding and absorbing additional moisture and will also protect the subbase from failure under traffic due to plastic flow. The abscissa shows the maximum amount of moisture that the soil may be expected to absorb when confined by the weight of this thickness of cover.

If the base material is questionable, the same procedure may be repeated, treating the base as the soil in question and the surface as the cover.

Should the expansion pressures be negligible, then the thickness would be determined by the resistance value at the degree of compaction which could be expected under field conditions. This degree of compaction can not be indicated by "density" determinations.

STATE OF CALIFORNIA DIVISION OF HIGHWAYS

MATERIALS & RESEARCH DEPARTMENT

PROCEDURE:

With a straightedge intersect Scale A at the value for R (as determined by the Stabilometer or some other substitute method) and Scale B at the traffic index for the total traffic load for the design life of the highway. The intersection of this line with Scale C is the thickness of gravel required to support the load (neglecting abrasion etc.). From this point intersect Scale D at the cohesiometer value of the surface. This line will intersect Scale E at the thickness of base and surface required to resist plastic flow of the basement soil.

When the thickness of the surface material is to be less than one-half that indicated on Scale C, correct cohesiometer value for use on Scale D as follows:

$$S = \frac{\text{pavement thickness}}{0.5 \times \text{Scale C Reading}} \times c$$

Where:

S = corrected cohesiometer value

c = original cohesiometer value.

THICKNESS DESIGN CHART
FOR BASE AND/OR PAVEMENT

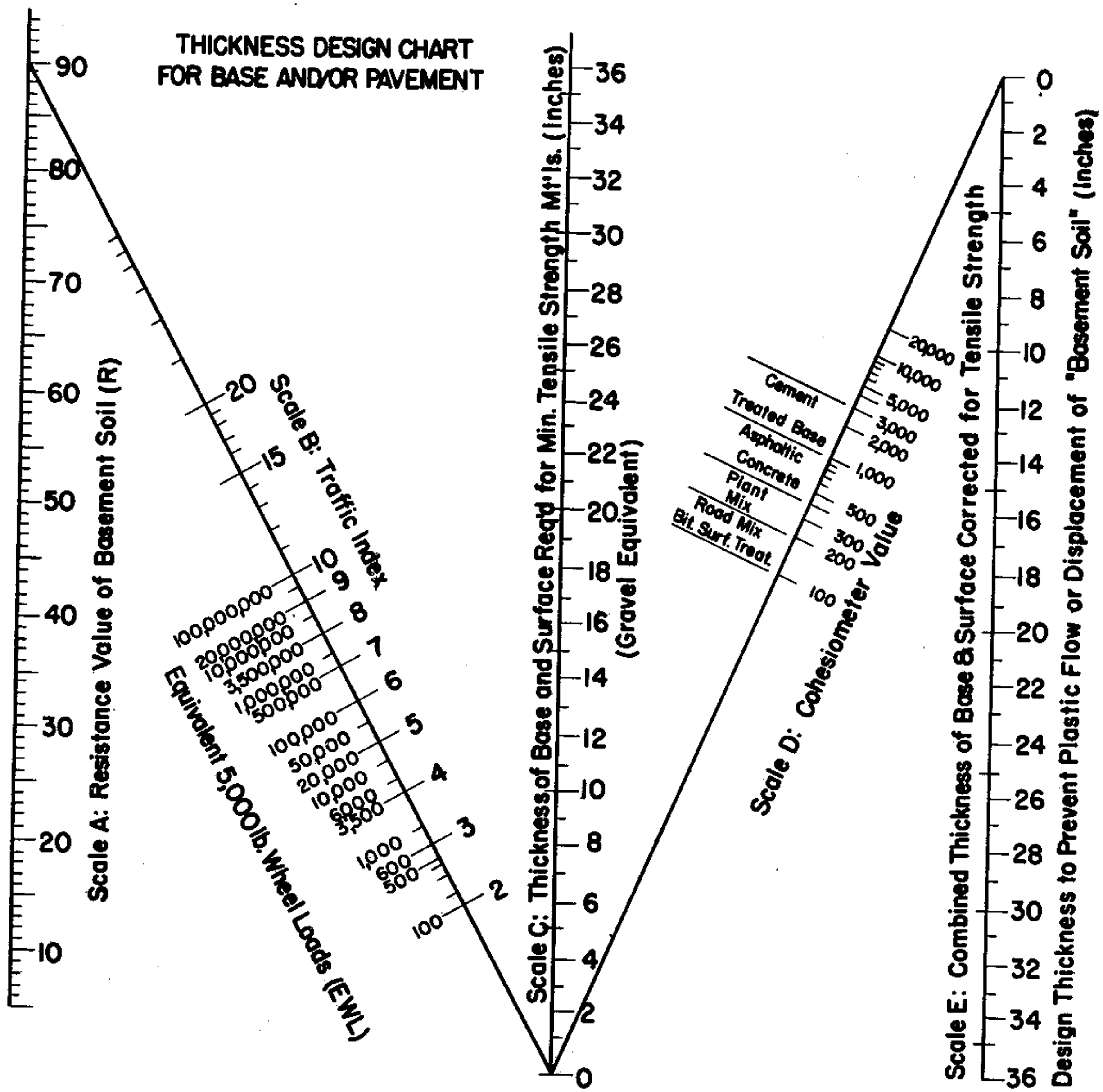
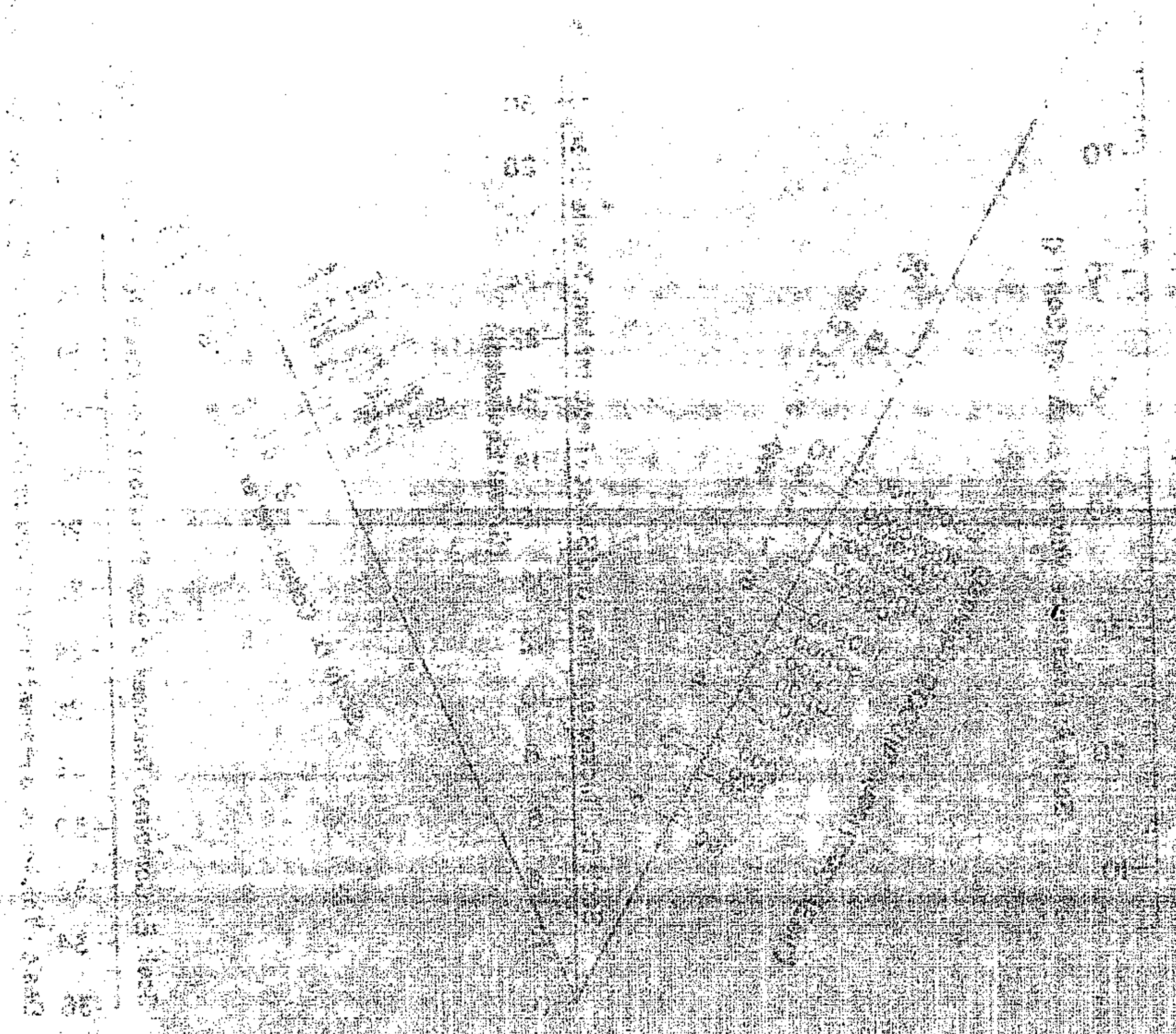
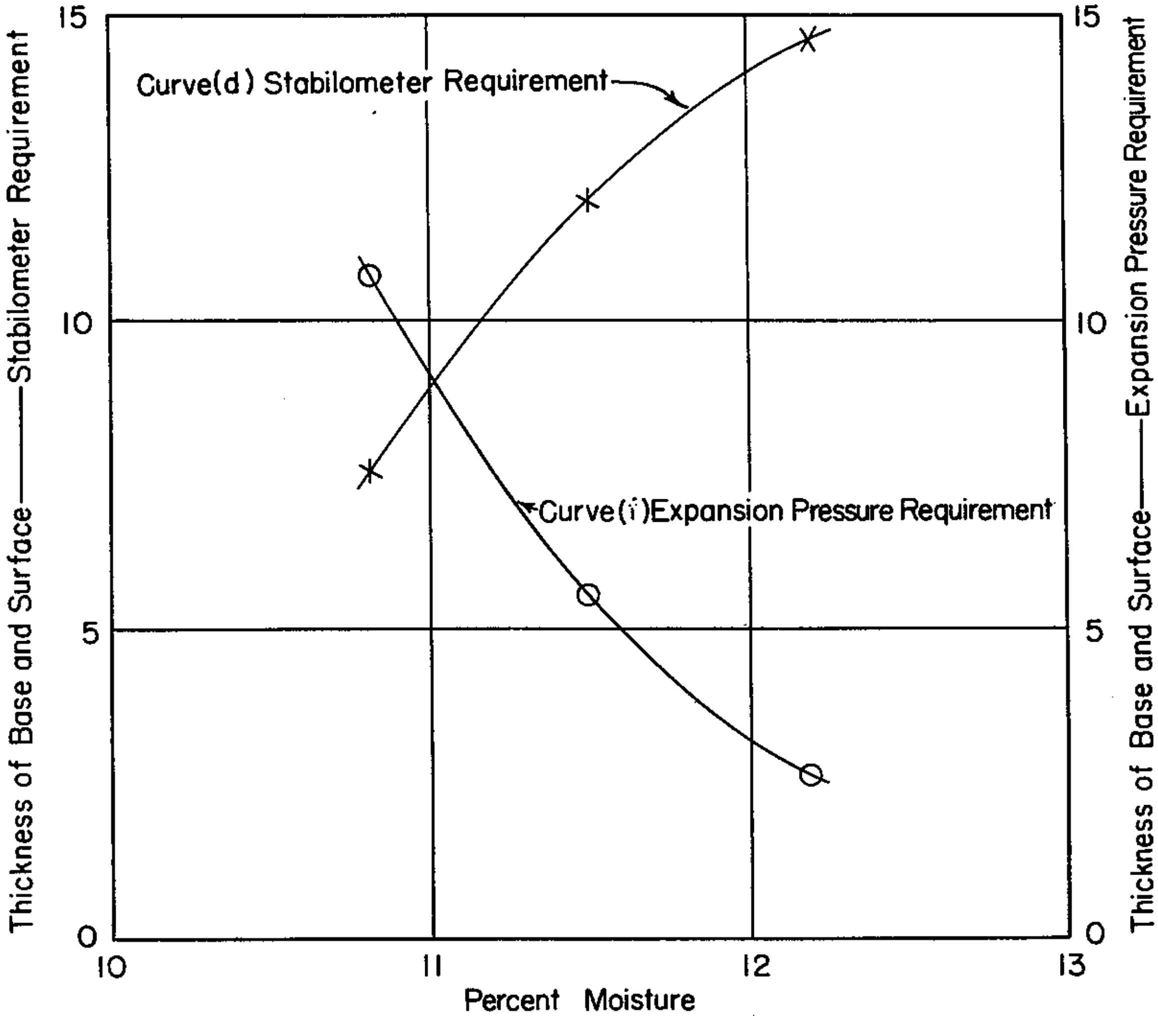


Fig. IV



shape. The intersection of ... Scale ... means of gravel required to support ... load ... Each line ... Scale B at the ... value of the surface ... Thickness ... Scale ...

When the thickness of the surface material is less than ... [illegible]



Curve(d) Stabilometer Requirement
Curve(i)Expansion Pressure Requirement
Thickness of Base and Surface——Stabilometer Requirement
Thickness of Base and Surface——Expansion Pressure Requirement
0
5
10
15
10
11
12
13
Percent Moisture

Fig. V

APPENDIX II

DEVELOPMENT OF A FORMULA FOR CALCULATING THE MOST PROBABLE PLANES OF PLASTIC SLIP UNDER CONDITION OF REPEATED LOADING

Failures of surfaces due to plastic movement of the subgrades occur under loads of a magnitude far less than a load which will cause stresses equal to the total frictional and cohesive resistance of the supporting soil. Accompanying de-deflections under loads causing ultimate failure are likewise small. Tests on experimental sections of flexible surfaces subjected to traffic (Brighton Test Track and Stockton Test Track) show that for a single-tired moving wheel of 6,000 pounds on a section which failed completely in less than 10,000 trips, the maximum initial deflection at the surface was only about .05 inches[1]; and for a single-tired moving wheel load of 40,000 pounds on a section which later failed completely with less than 10,000 repetitions of this load, the maximum initial deflection was about .15 inches.[2]

Under these small movements it is evident that, although the road is failing gradually under many load repetitions, at any one load application, the stresses are not sufficient to overcome the ultimate frictional and cohesive resistance of the soil.

To analyze the failure, it is necessary to compare the stresses with the strengths of all planes through the various points in the stressed soil and then determine along which planes the material is most likely to slip.

It has been common practice when dealing with static or fixed loads, such as are associated with embankments and retaining walls, to consider that the most dangerous plane at any point is where the difference between strength and stress is a minimum. In our case of repeated loading, a consideration of probabilities immediately tells us this would not be applicable. For example, it would mean that should the material be stressed to 5 psi. where the ultimate strength is 10 psi it would be as likely to plastic movement as when stressed to 95 psi where the ultimate strength is 100 psi.

[1]Brighton Test Track, 1940
[2]Stockton Test Track, 1942

Let us, therefore, state the following hypothesis of plastic movement under a repeated load: "The most probable plane of maximum plastic shearing strain will be that plane on which the stress is the greatest percentage of the ultimate strength; which may be expressed as a linear function of the unit cohesion, angle of friction, and normal pressure."

Let the diagram, Figure (a), represent any point in the soil within the influence of the wheel load. The plane of the paper is at right angles to the line of traffic. σ_1 and σ_2 represent the major and minor principal stresses, respectively, and we denote by p_α and S_α the normal and shearing stresses respectively, upon any plane through the point parallel to the line of traffic and making an angle α with the direction of σ_2

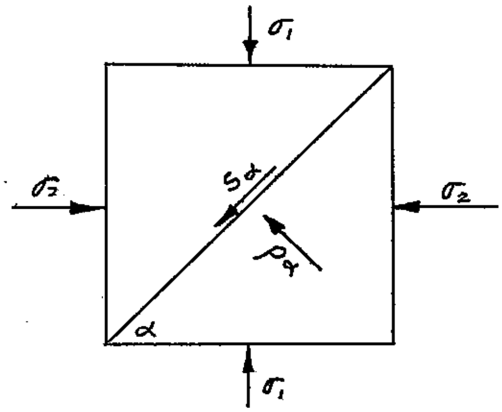


Fig (a)

Then:

$$p_\alpha = \frac{\sigma_1 + \sigma_2}{2} + \frac{\sigma_1 - \sigma_2}{2} \cos 2\alpha \qquad (1)$$

$$S_\alpha \quad \frac{\sigma_1 - \sigma_2}{2} \sin 2\alpha \qquad (2)$$

Assuming that the ultimate shearing resistance may be expressed by the linear function $C + p_\alpha \tan \phi$, then by our hypothesis the angle of most probable plastic movement will be where $\frac{S_\alpha}{C + p_\alpha \tan \phi}$ is a maximum (3)

By substituting (1) and (2) into (3), differentiating, equating to zero and solving for α, the following formula is obtained

$$\text{Cos. } 2\alpha = \frac{\sigma_2 - \sigma_1}{\frac{2C}{\tan\phi} + \sigma_1 + \sigma_2} \qquad (4)$$

Where α' is the angle of the most probable plane of plastic slip.

Had we used the criterion that the difference between the stress and strength ($C + p_\alpha \tan\phi - S_\alpha$) be a minimum, then by the same operations we would have obtained the familiar expression $\alpha'' = 45 + \frac{\phi}{2}$

(5)

Equation (4) and (5) become identical when the shearing stress equals the ultimate shearing resistance. In (4) α'

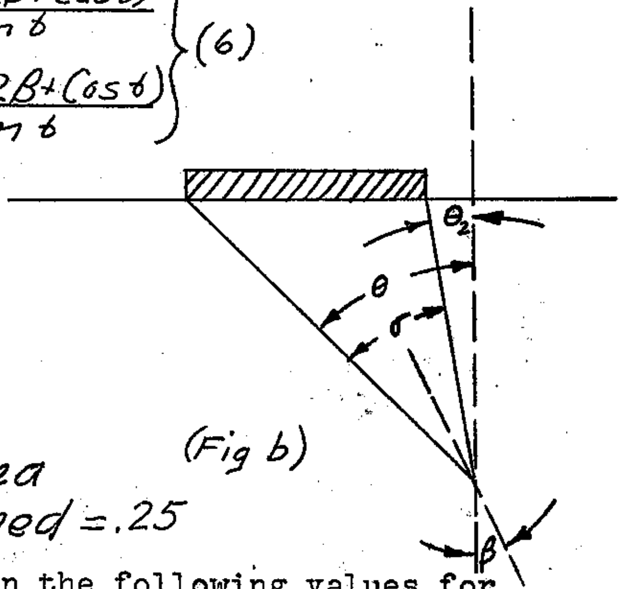
approaches 45° when c is large compared to Tan ϕ. In (5) equals 45° when ϕ = o

APPLICATION TO THE CASE OF A STRIP LOADING

Adding the stresses due to the weight of the soil to those due to the strip loading* by the method of superposition, we obtain the following formulas for the vertical, horizontal, and shear stresses, respectively.

$$\left.\begin{aligned} \sigma_z &= \frac{q}{\pi}\left[\delta + \sin\delta \cos 2\beta\right] + \frac{wa(\cos 2\beta + \cos\delta)}{2 \sin\delta} \\ \sigma_x &= \frac{q}{\pi}\left[\delta - \sin\delta \cos 2\beta\right] + \frac{wa(\cos 2\beta + \cos\delta)}{6 \sin\delta} \end{aligned}\right\} (6)$$

$$T_{xz} = \frac{q}{\pi}(\sin\delta \sin 2\beta)$$



(Fig b)

Where:
q = Unit Load
$\delta = \theta_1 - \theta_2$ (See Fig b)
$2\beta = \theta_1 + \theta_2$ (See Fig b)
w = Unit Weight of Soil
a = Width of Loaded Area
Poisson's Ratio assumed = .25

From equations (6) we obtain the following values for the major and minor principal stresses:

$$\left.\begin{aligned} \sigma_1 &= \frac{q}{\pi}\Big[\delta + \frac{\pi w a(\cos 2\beta + \cos\delta)}{3q \sin\delta} \\ &\quad + \sqrt{\sin^2\delta + \pi wa \cos 2\beta(\cos 2\beta + \cos\delta) + \pi^2 w^2 a^2(\cos 2\beta + \cos\delta)^2}\,\Big] \\ \sigma_2 &= \frac{q}{\pi}\Big[\delta + \frac{\pi wa(\cos 2\beta + \cos\delta)}{3q \sin\delta} \\ &\quad - \sqrt{\sin^2\delta + \pi wa \cos 2\beta(\cos 2\beta + \cos\delta) + \pi^2 w^2 a^2(\cos 2\beta + \cos\delta)^2} \end{aligned}\right\} (7)$$

*For treatment of strip loading see Terzaghi. Theo. Soil Mech., 1943, Page 377.

And the principal directions are found from the equation

$$Tan\, 2\theta = \frac{Sin\, 2\beta}{Cos\, 2\beta + \frac{\pi w a\, (Cos 2\beta + Cos\, \delta}{6q\, Sin^2 \delta}}$$

where θ = the angle made with the vertical.

Substituting the values for σ_1 and σ_2 equations (7) into equation (4), we obtain the equation

$$Cos\, 2\alpha' = \frac{Sin^2 \delta + \frac{\pi w a\, Cos 2\beta\, (Cos 2\beta + \delta)}{3q} + \frac{\pi^2 w^2 a^2 (Cos\, 2\beta + Cos\, \delta)^2}{36\, q^2\, Sin^2 \delta}}{\frac{\pi c}{q\, Tan\, \phi} + \delta + \frac{\pi w a\, (Cos\, 2\beta + Cos\, \delta)}{3q\, \; Sin\, \delta}}$$

from which we can determine the directions of the surfaces of most probable slip. α' is the angle made with the principal directions. $\alpha' + \theta - 90$ will be the angle made with the vertical.

Figure 23 shows the pattern of slip surfaces for a wheel load of approximately 6,000 pounds over a soil having little cohesion ($\frac{\pi c}{q\, tan\, \phi} = 0$). Figure 24 shows the same except that the weight of the soil has been neglected. Figures 24, 25 and 26 show the patterns of slip surfaces for different values of $\frac{\pi c}{q\, Tan\, \phi}$